

Chapter 8 Specialized Repairs

8-1. Lock Wall Rehabilitation

Approximately one-half of the Corps' navigation lock chambers were built prior to 1940. Consequently, the concrete in these structures does not contain intentionally entrained air and is therefore susceptible to deterioration by freezing and thawing. Since more than 75 percent of these older structures are located in the U.S. Army Engineer Divisions, North Central and Ohio River, areas of relatively severe exposure to freezing and thawing, it is not surprising that many of these structures exhibit significant concrete deterioration. Depending upon exposure conditions, depths of concrete deterioration can range from surface scaling to several feet. The general approach in lock wall rehabilitation has been to remove 0.3 to 0.9 m (1 to 3 ft) of concrete from the face of the lock wall and replace it with new portland-cement concrete using conventional concrete forming and placing techniques (McDonald 1987b). Other approaches that have been used include shotcrete, preplaced-aggregate concrete, and precast concrete stay-in-place forms. Also, a variety of thin overlays have been applied to lock walls.

a. Cast-in-place concrete. The economics of conventional cast-in-place concrete replacement compared to other rehabilitation techniques usually depends on the thickness of the concrete section to be replaced. For sections in the range of 152 to 305 mm (6 to 12 in.), both formed and nonformed techniques such as shotcrete are economically competitive. When the thickness of the replacement section exceeds 305 mm (12 in.), conventional formwork and concrete replacement are generally more economical. Conventional cast-in-place concrete has several advantages over other rehabilitation materials. It can be proportioned to simulate the existing concrete substrate, thus minimizing strains due to material incompatibility; proper air entrainment in the replacement concrete can be obtained by use of admixtures to ensure resistance to cycles of freezing and thawing; and materials, equipment, and personnel with experience in conventional concrete application are readily available in most areas.

(1) Surface preparation. Removal of deteriorated concrete is usually accomplished by drilling a line of small-diameter holes along the top of the lock wall parallel to the removal face, loading the holes with light charges of explosive (usually detonating cord), cushioning the charges by stemming the holes, and detonating the

explosive with electric blasting caps. Blasting plans (holes spacing, size of charges, necessity for delays in detonation of charges, etc.) are developed based on results of previous work or test blasts. After the existing concrete is removed, the lock walls are sounded to locate any areas of loose or deteriorated concrete extending beyond the removal line. Such concrete is removed by chipping, grinding, or water blasting. The bonding surface must be clean and free of materials that could inhibit bond.

(2) Concrete anchors. Dowels are normally used to anchor the new concrete facing to the existing concrete walls and to position vertical and horizontal reinforcing steel in the concrete facing. Design criteria for use in design of dowels for anchoring relatively thin sections (less than 0.8 m (2.5 ft)) of cast-in-place concrete facing were developed based on laboratory and field tests (Liu and Holland 1981).

(a) Dowels should be No. 6 deformed reinforcing bars conforming to ASTM A 615. Typical dowel spacing is 1.2 m (4 ft) center to center in both directions, except that a maximum dowel spacing of 0.6 m (2 ft) center to center may be specified in the vicinity of local openings and recesses, and along the perimeters of monoliths. The band width of this dowelling should be at least 0.6 m (2 ft).

(b) If the average compressive strength of three drilled cores obtained from the existing concrete as determined in accordance with ASTM C 42 (CRD-C 27) is less than 21 MPa (3,000 psi), the embedment length should be determined by conducting field pullout tests. For existing concrete with an average compressive strength equal to or greater than 21 MPa (3,000 psi), dowel embedment length should be a minimum of 15 times the nominal diameter of the dowel unless a shorter embedment length can be justified through field pullout tests. Dowels should be embedded in holes drilled with rotary-percussive equipment. The drill holes must be clean and free of materials that can inhibit bond. Either a nonshrink grout conforming to ASTM C 1107 (CRD-C 621) or an epoxy resin-based bonding system conforming to ASTM C 881 (CRD-C 595), Type I, or other approved bonding systems should be used as the bonding agent. The embedment length for dowels in the new concrete facing should be determined in accordance with ACI 318.

(c) A minimum of 3 dowels per 1,000 to be installed should be field tested with the testing dispersed over the entire surface area to receive dowels. Test procedures should be similar to those described by Liu and Holland

(1981). The embedment length is considered adequate when the applied pullout load is equal to or greater than the calculated yield strength for the dowel in all tests.

(d) Prepackaged polyester-resin grout has been used to embed the anchors on most projects, and field pullout tests on anchors installed under dry conditions indicate this to be a satisfactory procedure.

(3) Reinforcement. Mats of reinforcing steel, usually No. 5 or 6 bars on 305-mm (12-in.) centers each way, are hung vertically on the dowels (Figure 8-1). Concrete cover over the reinforcing is usually 100 or 127 mm (4 or 5 in.). In some cases, the reinforcing mat, wall armor, and other lock wall appurtenances are installed on the form prior to positioning the form on the face of the lock wall (Figure 8-2).

(4) Concrete placement. Once the reinforcement and formwork are in position, replacement concrete is placed by pumping or discharging it directly into hoppers fitted with various lengths of flexible pipe commonly known as elephant trunks. Lift heights varying from 1.5 m (5 ft) to full face of approximately 15 m (50 ft) have been used. Normally, concrete is placed on alternating monoliths along a lock chamber wall. Generally, forms are removed 1 to 3 days following concrete placement and a membrane curing compound applied to formed concrete surfaces.

(5) Performance. One of the most persistent problems in lock wall rehabilitation resulting from use of this

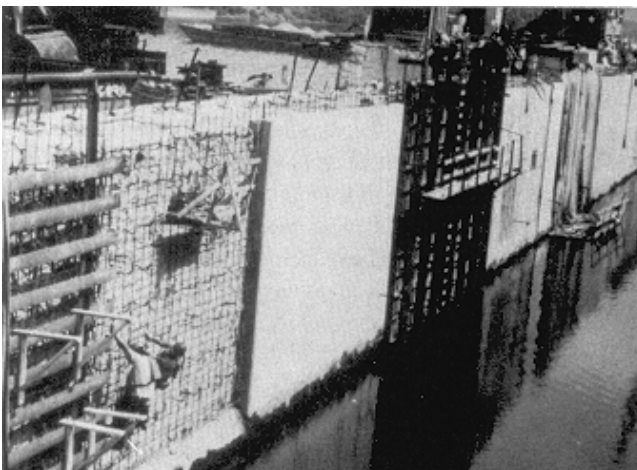
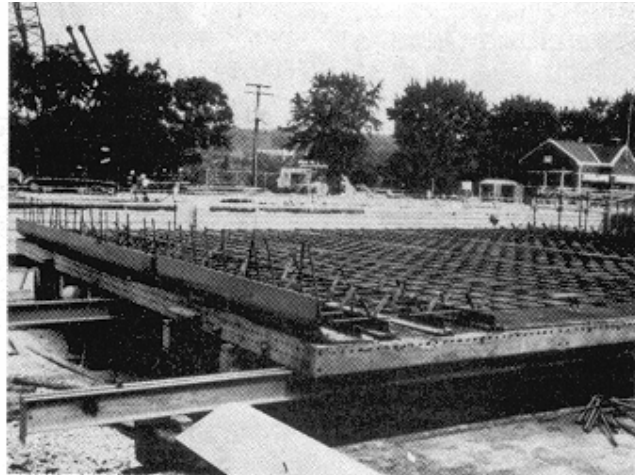
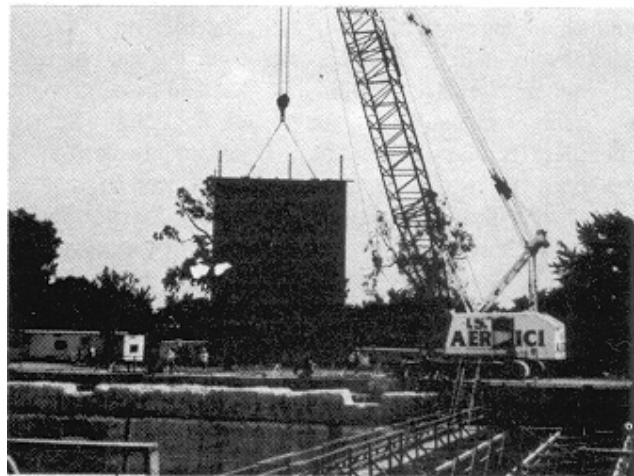


Figure 8-1. Placing reinforcing steel and horizontal armor prior to concrete placement, Locks and Dam No. 3, Monogahela River



a. Wall armor and concrete reinforcement in place on the form



b. Concrete forming being placed in position on the lock wall

Figure 8-2. Lock wall rehabilitation, Brandon Rock Lock

approach is cracking in the replacement concrete. These cracks, which generally extend completely through the conventional replacement concrete, are attributed primarily to the restraint provided through bond of the new concrete to the stable mass of existing concrete. As the relatively thin layer of resurfacing concrete attempts to contract as a result of shrinkage, thermal gradients, and autogenous volume changes, tensile strains develop in the replacement concrete. When these strains exceed the ultimate tensile strain capacity of the replacement concrete, cracks

develop. The lock monolith on the left in Figure 8-3 was resurfaced with a 457-mm (18-in.) overlay of new concrete. As a result of the restraint provided by the existing concrete, extensive cracking developed. The monolith on the right was completely removed and reconstructed in 1.5-m (5-ft) lifts with the same concrete materials and mixture proportions. Without an existing concrete substrate to provide restraint, no cracking occurred.

(a) A finite element analysis of a typical lock wall resurfacing (Norman, Campbell, and Garner 1988) also demonstrated the effect of restraint by the existing concrete. Under normal conditions of good bond between the two concretes, stress levels sufficient to cause cracking in the replacement concrete developed within about 3 days (Figure 8-4a). In comparison, a bond breaker at the interface resulted in almost no stress in the replacement concrete. The analysis also indicated that shrinkage was a significant factor (Figure 8-4b). Even the lower bound shrinkage resulted in tensile stresses in excess of 2.1 MPa (300 psi).

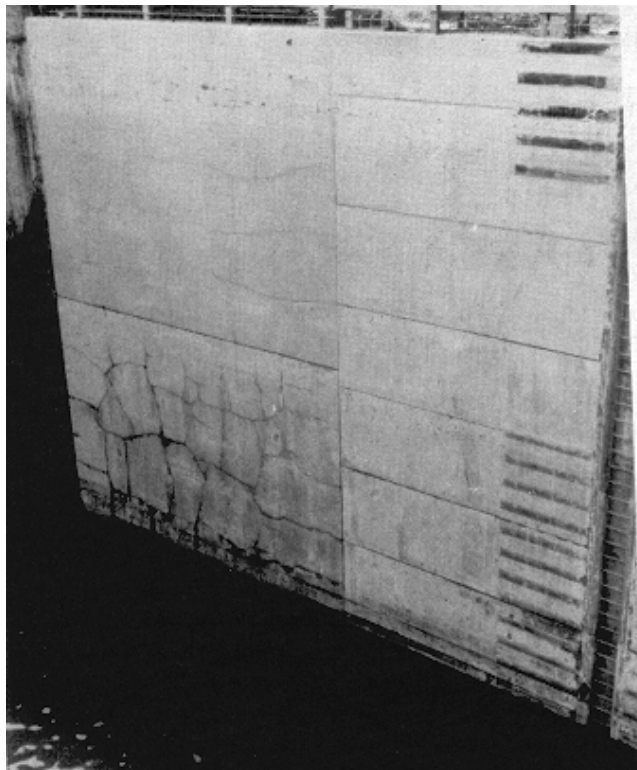


Figure 8-3. Concrete cracking in resurfaced monolith (left) compared to reconstructed monolith, Lock No. 1, Mississippi River

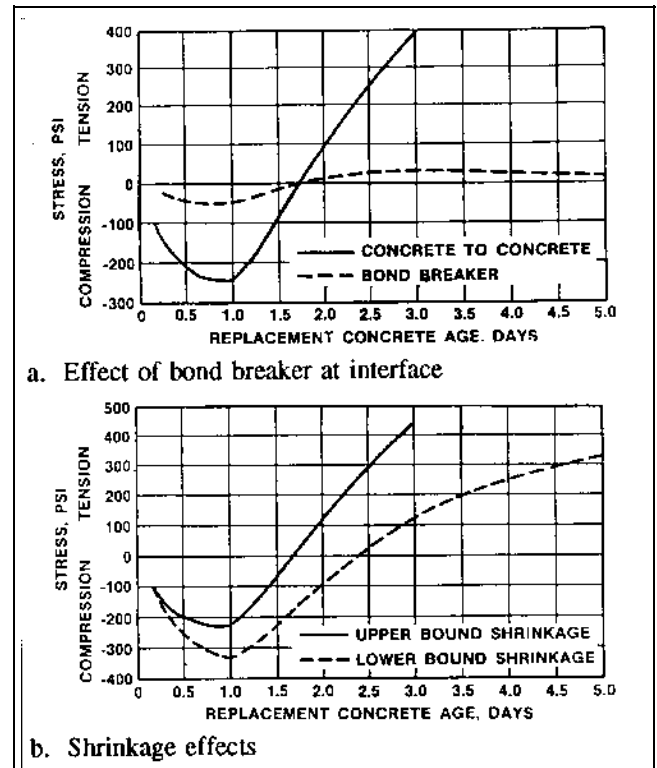


Figure 8-4. Results of a finite element analysis of a typical lock wall resurfacing

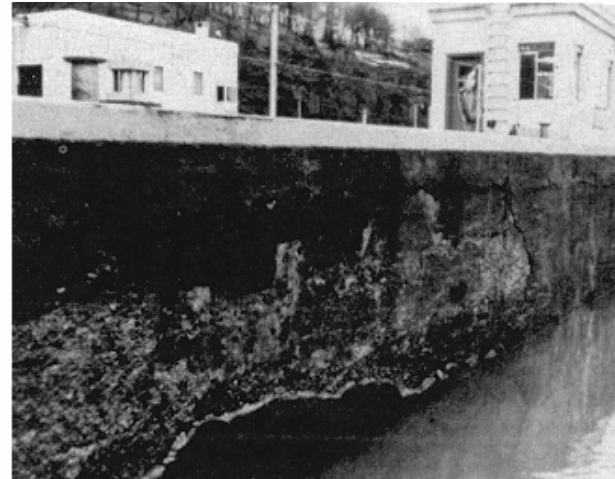
(b) CEWES-SC recommendations to minimize shrinkage and install a bond breaker (Hammons, Garner, and Smith 1989) were implemented by the U.S. Army Engineer District, Pittsburgh, during the rehabilitation of Dashields Locks, Ohio River. An examination of the project indicated that cracking of concrete placed during 1989 was significantly less than that of concrete placed during the previous construction season prior to implementation of CEWES-SC recommendations. Also, resurfacing of the lock walls at Lock and Dam 20, Mississippi River, resulted in significantly less cracking in the conventional concrete than was previously experienced at other rehabilitation projects within the U.S. Army Engineer District, Rock Island. The reduced cracking was attributed to a combination of factors including lower cement content, larger maximum size coarse aggregate, lower placing and curing temperatures, smaller volumes of placement, and close attention to curing (Wickersham 1987). Preformed horizontal contraction joints 1.5 m (5 ft) on center were effective in controlling horizontal cracking in the rehabilitation of Lock No. 1, Mississippi River (McDonald 1987b).

b. Shotcrete.

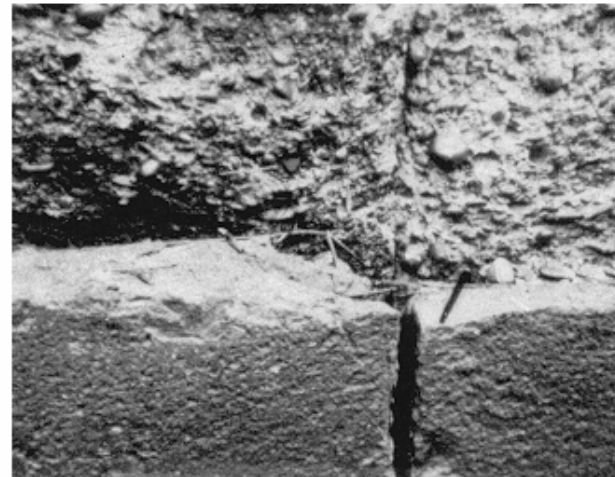
(1) General considerations. For repair of sections less than 152 mm (6 in.) thick, shotcrete is generally more economical than conventional concrete because of the saving in forming costs. Properly applied shotcrete is a structurally adequate and durable material, and it is capable of excellent bond with concrete and other construction materials. These favorable properties make shotcrete an appropriate selection for repair in many cases. However, there are some concerns about the use of shotcrete to rehabilitate old lock walls. The resistance of shotcrete to cycles of freezing and thawing is generally good despite a lack of entrained air. This resistance is attributed in part to the low permeability of properly proportioned and applied shotcrete which minimizes the ingress of moisture thus preventing the shotcrete from becoming critically saturated. Consequently, if the existing nonair-entrained concrete in a lock wall behind a shotcrete repair never becomes critically saturated by migration from beneath or behind the lock wall, it is likely that such a repair will be successful. However, if moisture does migrate through the lock wall and the shotcrete is unable to permit the passage of water through it to the exposed surface, it is likely that the existing concrete will be more fully saturated during future cycles of freezing and thawing. If frost penetration exceeds the thickness of the shotcrete section under these conditions, freeze-thaw deterioration of the existing nonair-entrained concrete should be expected.

(2) Performance.

(a) The river chamber at Emsworth Locks and Dam, opened to traffic in 1921, was refaced with shotcrete in 1959. By 1981 the shotcrete repair had deteriorated to the stage shown in Figure 8-5. Spalling appeared to have originated in the upper portion of the wall where the shotcrete overlay was relatively thin and surface preparation minimal. Deterioration apparently propagated down the wall to a point where the shotcrete was of sufficient thickness (approximately 100 mm (4 in.)) to contain dowels and wire mesh. Horizontal cores from the chamber walls showed the remaining shotcrete to be in generally good condition (Figure 8-6a). However, the original concrete immediately behind the shotcrete exhibited significant deterioration, probably caused by freezing and thawing. Cores of similar concrete from the land chamber which did not receive a shotcrete overlay were in generally good condition from the surface inward (Figure 8-6b). This appears to be an example of an overlay contributing to the saturation of the original concrete with increased deterioration resulting from freezing and thawing.



a. Overall view



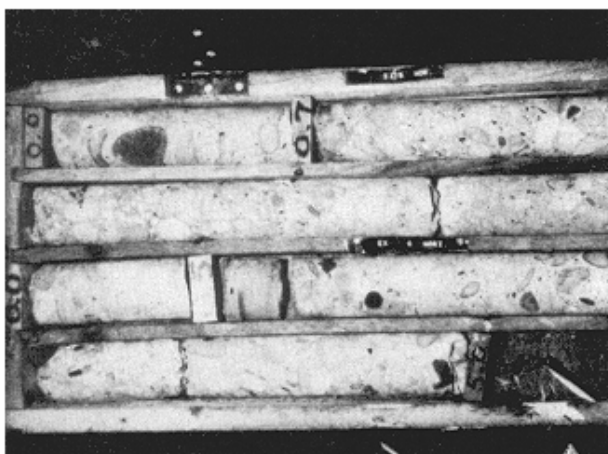
b. Closeup view

Figure 8-5. Spalling of shotcrete used to reface river chamber wall, Emsworth Locks and Dam

(b) Anchored and reinforced shotcrete was used to reface wall areas of both the Davis and Sabin Locks at Sault Ste. Marie, Michigan. After 30 to 35 years in service, the shotcrete appeared to be in generally good condition. However, cores taken from the lock walls in 1983 showed deterioration of the nonair-entrained concrete immediately behind the shotcrete in several cases (Figure 8-7a). In comparison, concrete in wall areas which were not refaced was in good condition (Figure 8-7b). Laboratory tests of selected cores indicated the permeability of the shotcrete to be approximately one-third that of the concrete. Also, the shotcrete contained



a. Portions of the core from two drill holes through river chamber wall which had been refaced with shotcrete



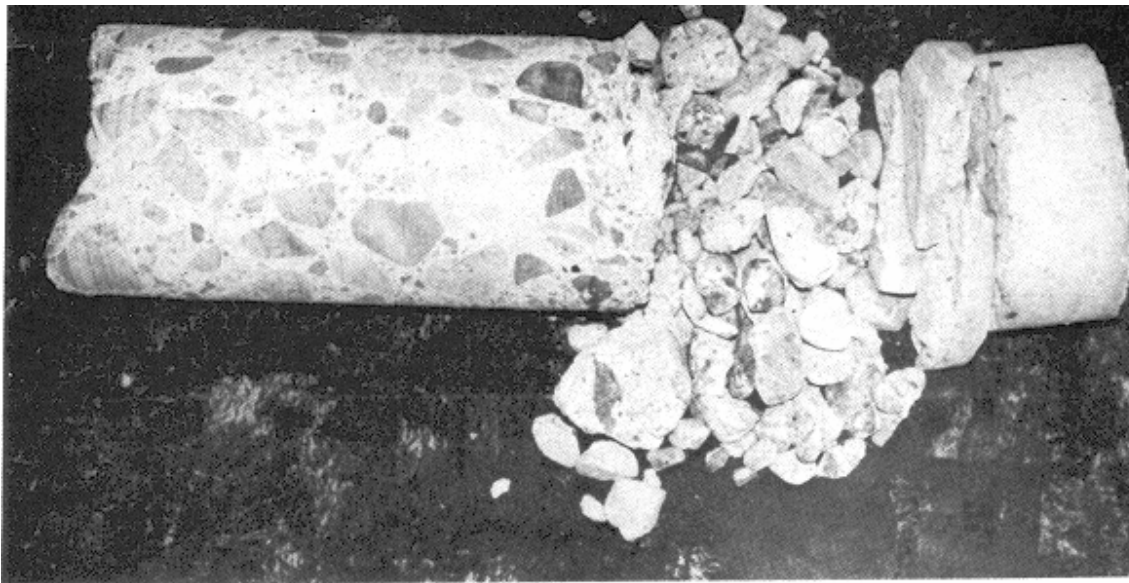
b. Portions of the core from two drill holes through land chamber wall which was not refaced

Figure 8-6. Comparison of cores taken horizontally from lock chamber walls, Emsworth Locks

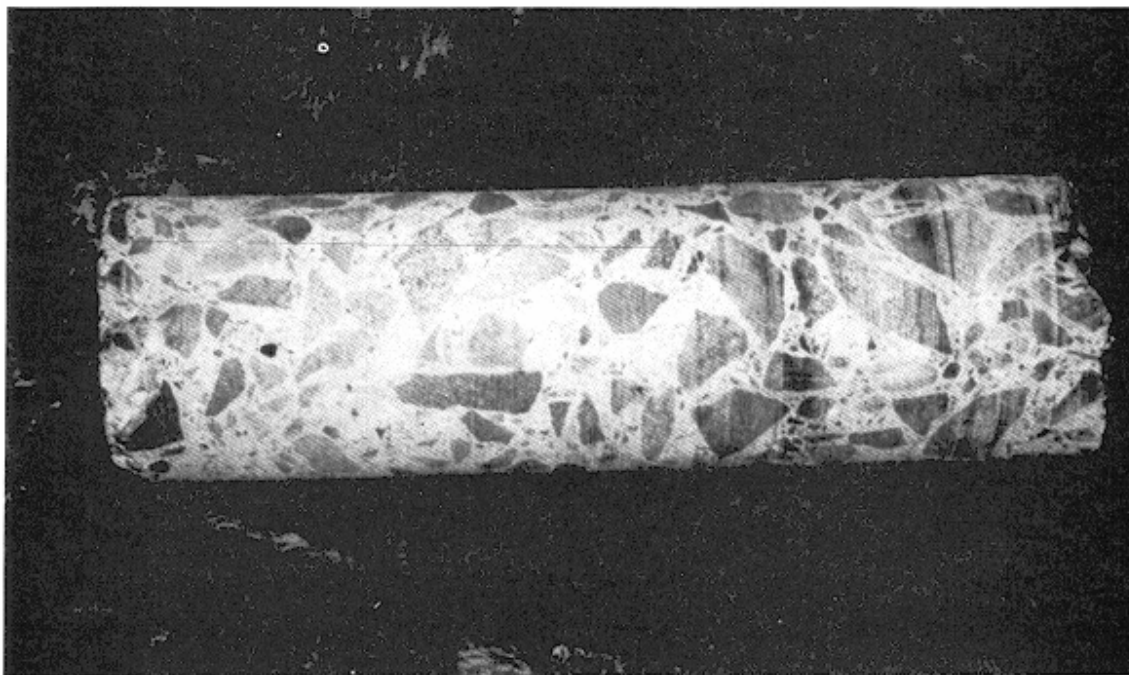
1.8-percent air voids compared to 0.9 percent in the concrete. In areas where moisture migration did not cause the concrete behind the shotcrete to become critically saturated during cycles of freezing and thawing, the shotcrete was well bonded to the concrete and there was no evidence of deterioration in either material. However, when cores from such areas were subjected to accelerated freeze-thaw testing in a saturated state, the concrete completely disintegrated while the shotcrete remained generally intact (Figure 8-8). Similar cores subjected to accelerated freeze-thaw testing in a dry state remained substantially intact (Figure 8-9).

(c) Portions of the chamber walls at Dresden Island Lock were repaired in the early 1950's with anchored and good condition after about 40 years in service. This good performance relative to other shotcrete repairs is attributed primarily to the thickness of the shotcrete overlay which apparently exceeded the depth of frost penetration. Three horizontal cores taken through the repaired walls in the mid-1970's indicated that the overlay is a minimum of 305 mm (12 in.) thick. The shotcrete-concrete interface was intact and the concrete immediately behind the shotcrete exhibited no deterioration. After more than 40 years in service, the average depth of deterioration in the unrepaired concrete walls as determined by petrographic examination was approximately 215 mm (8-1/2 in.). Assuming that the depth of frost penetration is approximately equal to the depth of deterioration, the thickness of shotcrete is about 50 percent greater than the depth of frost penetration. Therefore, the concrete behind the shotcrete would not be expected to exhibit freeze-thaw deterioration even though it may have been critically saturated. Air-void data determined according to ASTM C 457 (CRD-C 42) indicated the shotcrete had about 3 percent total air with approximately 2 percent of it in voids small enough to be classified as useful for frost resistance. The air-void spacing factors ranged from 0.25 to 0.36 mm (0.010 to 0.014 in.). While these values are larger than is desirable (0.20 mm (0.008 in.) is considered the maximum value for air-entrained concrete), they may have imparted some frost resistance. Also, there were no large voids or strings of voids resulting from lack of consolidation such as have been observed with other shotcrete specimens.

(d) The concrete in the walls of Lower Monumental Lock has an inadequate air-void system to resist damage caused by freezing and thawing while critically saturated. After approximately 10 years in service, the concrete had deteriorated to the point that the aggregate was exposed. After laboratory tests and a full-scale field test which demonstrated that a 10-mm (3/8-in.)-thick latex-modified fiberglass-reinforced spray-up coating had the potential to meet repair requirements, which included time constraints on the lock downtime that precluded conventional concrete replacement (Schrader 1981), the lock walls were repaired (Figure 8-10) in 1980. In March 1983 several small isolated areas of debonded coating (less than 1 percent of the total area) were identified in the lock. The coating in these locations was removed and the areas resprayed similar to the original repair. Also, a fairly large debonded zone in monolith 11 was repaired. A September 1983 inspection reported the original coating to



a. Concrete deterioration immediately behind shotcrete overlay



b. Concrete not refaced with shotcrete

Figure 8-7. Comparison of cores taken from lock chamber walls, Davis and Sabin Locks

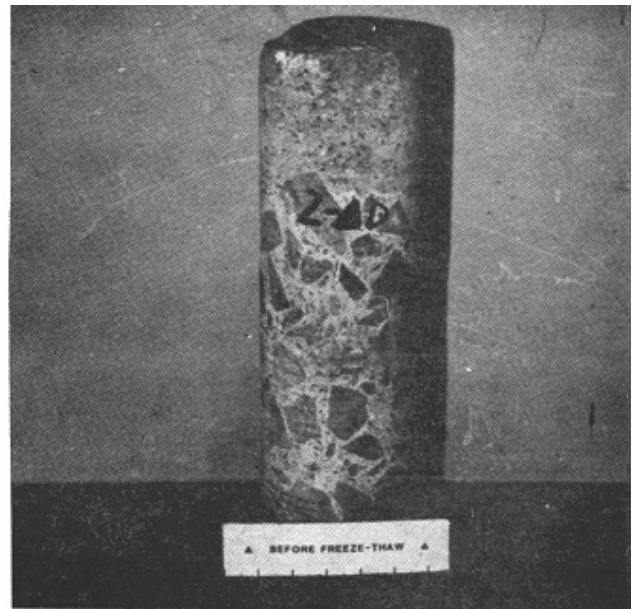


a. Before freezing and thawing

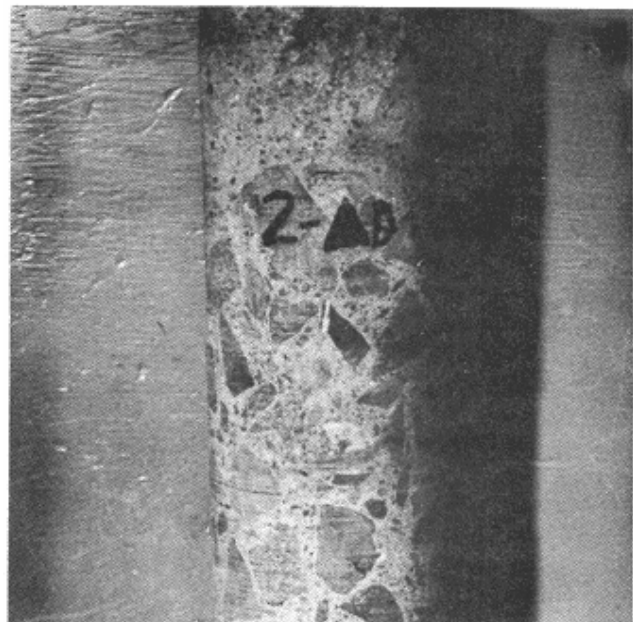


b. After freezing and thawing

Figure 8-8. Combination shotcrete and concrete core before and after 35 cycles of freezing and thawing in a saturated condition



a. Before freezing and thawing

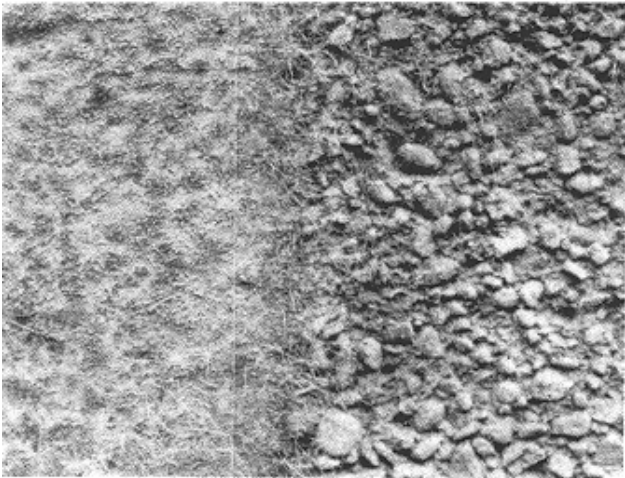


b. After freezing and thawing

Figure 8-9. Combination shotcrete and concrete core before and after 35 cycles of freezing and thawing in a dry condition



a. Applying the coating with equipment that simultaneously sprays the mortar from one nozzle while it chops and blows the fiber from a cutterhead



b. Typical appearance of the lock wall before and after repair

Figure 8-10. Application of coating to lock wall, Lower Monumental Lock

be in generally good condition. However, the coatings applied earlier in 1983 had all failed after only 6 months in service. The major difference in the two repairs was the type of latex used. Saran was used in the original work, whereas styrene butadiene was used in the repairs which failed. It was also reported that delaminations were common where latex spray-up material had been applied over hardened latex from a previous coating. One possibility for this failure is that the high-pressure water jet did not properly clean the surface of latex film prior to application of the subsequent layer. After approximately 5 years in service, large pieces of the spray-up coating began to fall from the lock walls. Failure occurred in the

concrete substrate immediately behind the bond line. Moisture migrating toward the surface from within the mass concrete was trapped at the interface because of the low water vapor transmission characteristics of the coating. Subsequent ice and hydraulic pressure caused the coating to debond (Schrader 1992).

c. *Preplaced-aggregate concrete.* Since drying shrinkage and creep occur almost exclusively in the cement paste fraction of concrete and since both phenomena are resisted by the aggregate, particularly if the coarse aggregate particles are in point-to-point contact, drying shrinkage and creep are both remarkably less for preplaced-aggregate concrete than for conventionally placed concrete. The reduction in drying shrinkage reduces the probability of cracking under conditions of restrained shrinkage, but it is at the expense of a reduced capability to relax concentrated stresses through creep. The dimensional stability of preplaced-aggregate concrete makes it attractive as a material for the rehabilitation of lock walls and appurtenant structures, particularly if it is successful in mitigating or eliminating the unsightly cracking commonly experienced with conventionally placed concrete. Its potential is further enhanced by the fact that it can be conveniently formed and placed underwater and that it can be grouted in one continuous operation so that there are no cold joints. Laboratory investigation has shown preplaced-aggregate concrete to be superior to conventionally placed, properly air-entrained concrete in resistance to freeze-thaw damage and impermeability (Davis 1960). Cores drilled through preplaced-aggregate concrete overlays have shown excellent bond with the parent concrete.

(1) Applications.

(a) Preplaced-aggregate concrete has been used extensively and effectively to construct or rehabilitate many different kinds of structures, but the technique is not one that many contractors have had experience with. Successful execution of preplaced-aggregate work requires a substantial complement of specialized equipment mobilized and operated by a seasoned crew working under expert supervision. This coupled with the fact that preplaced-aggregate concrete requires stronger and tighter formwork generally results in higher bid prices, as much as one-third higher than conventional concrete according to current bid prices. Therefore, if preplaced-aggregate concrete is the desired repair material, it must be specified uniquely and not as an alternate.

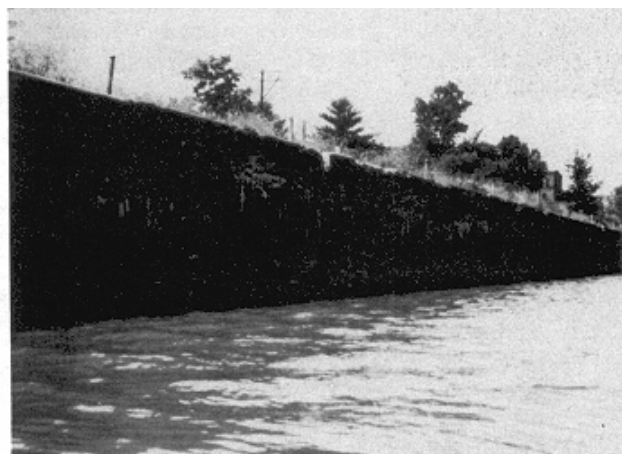
(b) Constructed during the period 1907-1910, the lock chamber walls at Lock No. 5, Monongahela River, required refacing in 1950. The plans called for removal of approximately 450 mm (18 in.) of old concrete from an area extending from the top of the lock walls to about 450 mm (18 in.) below normal pool elevation and the refacing of this area with reinforced concrete. Specifications required that the concrete have a minimum 28-day compressive strength of 24 MPa (3,500 psi) and provided that concrete could be placed by either conventional or preplaced aggregate methods. Also, it was required that one of the two lock chambers be open to navigation at all times. The low bid (\$85,000) was submitted by Intrusion-Prepakt Co. to whom the contract was awarded. The low bid was almost \$60,000 lower than the second lowest bid. The contractor elected to perform the work without constructing cofferdams. This method necessitated the removal of existing concrete below pool elevation "in the wet," and required that the new concrete below pool elevation be placed behind watertight bulkheads to exclude pool water from the spaces to be filled with concrete. The contractor elected to use the preplaced-aggregate method for concrete placement. Details of the repair operation were reported by Minnotte (1952).

(c) Preplaced-aggregate concrete was used in 1987 to resurface the lock chamber walls at Peoria Lock (Mech 1989). Following removal of a minimum of 305 mm (12 in.) of concrete, the walls were cleaned with high-pressure water, anchors and reinforcing steel were installed, and forms were positioned on individual monoliths. Typical resurfaced areas were about 3 m (10 ft) high and 12 m (40 ft) wide. The forms were filled with water to reduce breakage of the coarse aggregate during placement. The forms were vibrated externally with handheld equipment during grout intrusion which took about 8 to 10 hr of pumping for each monolith. Forms on two of the approximately 30 monoliths had to be reset because anchors holding the forms to the lock wall failed.

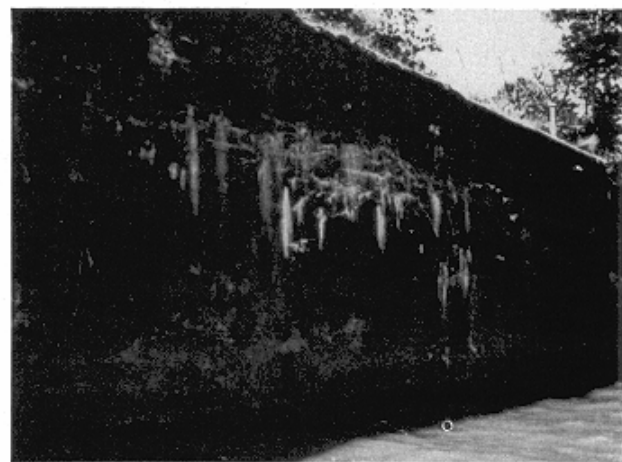
(2) Performance.

(a) During the Lock No. 5 repair, preplaced-aggregate concrete test cylinders were made by filling steel molds with coarse aggregate and pumping the mortar mixture into the aggregate through an insert in the base of the molds. Compressive strengths of these 152- by 305-mm (6- by 12-in.) specimens ranged from 20 to 30 MPa (2,880 to 4,300 psi) with an average strength of 26 MPa (3,800 psi). The average compressive strength of cores, drilled from the refacing concrete 1 year later, was 37 MPa (5,385 psi).

(b) Lock No. 5 was removed from service and, with the exception of the land wall, razed in conjunction with the construction of Maxwell Lock and Dam in 1964. A visual examination of the remaining wall in July 1985 showed that the preplaced-aggregate concrete had some cracking and leaching (Figure 8-11) but overall appeared to be in generally good condition after 35 years exposure. This repair demonstrates that the preplaced-aggregate method of concrete placement is a practical alternative to conventional methods for refacing lock walls.



a. Overall view



b. Closeup view

Figure 8-11. Condition of lock chamber wall 35 years after refacing with preplaced-aggregate concrete

(c) Although the repair at Peoria Lock was generally successful, the preplaced-aggregate concrete exhibited some cracking. Some of the cracks could be traced to areas where the contractor had problems with the soaker system used for moist curing. High temperatures during this period ranged between 35 and 38 °C (95 and 100 °F). Some bug holes on the concrete surface were attributed to the method used to vibrate the forms and changing the grout flow requirement from 18 sec to 26 sec.

(d) The bid price for 466 cu m (610 cu yd) of preplaced-aggregate concrete at Peoria Lock was \$1,250 per cu m (\$960 per cu yd, which included form work, aggregate, placement, grouting operation, and finishing). In comparison, typical bid prices for conventional concrete used on other rehabilitation projects in the U.S. Army Engineer District, Rock Island, have been in the range of \$590 to \$850 per cu m (\$450 to \$650 per cu yd) and higher for difficult construction situations.

d. Thin overlays. A variety of thin overlays have been applied to lock walls (McDonald 1987b). In most cases, they have been used in areas where the depth of deterioration was minimal, and apparently the intent was to protect the existing concrete or to improve the appearance of the structure. There has been little, if any, concrete removal associated with these applications. Thin overlays have had very little success with a number of failures during the first winter after application. Such overlays are particularly susceptible to damage by barge impact and abrasion.

e. Precast concrete. Compared with cast-in-place concrete, precasting offers a number of advantages including minimal cracking, durability, rapid construction, reduced future maintenance costs, and improved appearance. Also, precasting minimizes the impact of adverse weather and makes it possible to inspect the finished product prior to its incorporation into the structure. Precast-concrete stay-in-place forms have been used successfully in a number of lock wall rehabilitation projects. In addition to significant reductions in the length of time a lock must be closed to traffic for rehabilitation, the precast-concrete stay-in-place forming systems has the potential to eliminate the need for dewatering a lock chamber during wall resurfacing (ABAM Engineers 1989). Additional information on applications of precast concrete in repair and replacement of a variety of civil works structures is given in Section 8-5.

(1) Development. A precast concrete stay-in-place form system for lock wall rehabilitation was developed as part of the REMR Research Program. The objectives of

this work were to develop a precast concrete rehabilitation system which provides superior durability with minimal cracking, accommodates all of the normal lock hardware and appurtenances, minimizes lock downtime, and can be implemented at a wide variety of project sites. To accomplish these goals, the system was required to satisfy a well-defined set of durability, functional, constructibility, and cost/schedule criteria (McDonald 1987a).

(a) A wide range of alternatives for achieving the design objectives was evaluated through a process of value engineering (ABAM Engineers 1987a). Based on this analysis, it was concluded that the most advantageous combination of design alternatives was a precast-quality concrete (minimum compressive strength of 45 MPa (6,500 psi)), conventionally reinforced, flat panel, horizontally oriented and tied to the lock wall (Figure 8-12). A typical panel was 165 mm (6-1/2 in.) thick, 1.8 m (6 ft) wide, 9.1 m (30 ft) long, and it weighed approximately 7 tons. The panels are tied to the lock monolith along the top and bottom edges with form ties designed to support the loads of the infill concrete placement.

(b) To demonstrate the constructibility of the system, eight panels of varying sizes were precast and installed on two one-half-scale simulated lock monoliths (Figure 8-13). The purpose of the demonstration was to evaluate the feasibility of the precast concrete stay-in-place forming system without the risk and investment of undertaking a full-scale lock rehabilitation. Results of this work demonstrated that the precast concrete stay-in-place forming system is a viable method for lock wall rehabilitation (ABAM Engineers 1987b). In addition to providing a concrete surface of superior durability with minimal cracking, the construction cost was very competitive with the cost of conventional cast-in-place concrete. The demonstration also identified a number of areas where the design and installation procedures could be enhanced thus reducing both the cost and schedule associated with the stay-in-place forming system. Development of the precast concrete stay-in-place forming system is summarized in a video report (McDonald 1988) which is available on loan from the CEWES Library.

(2) Applications. To date, the precast concrete stay-in-place forming system has been used to rehabilitate four lock chambers, and contracts for two additional projects have been awarded. In addition, precast concrete panels have been used to overlay the back side of the river wall of two locks. Detailed descriptions of these applications (McDonald and Curtis in preparation) are summarized in the following.

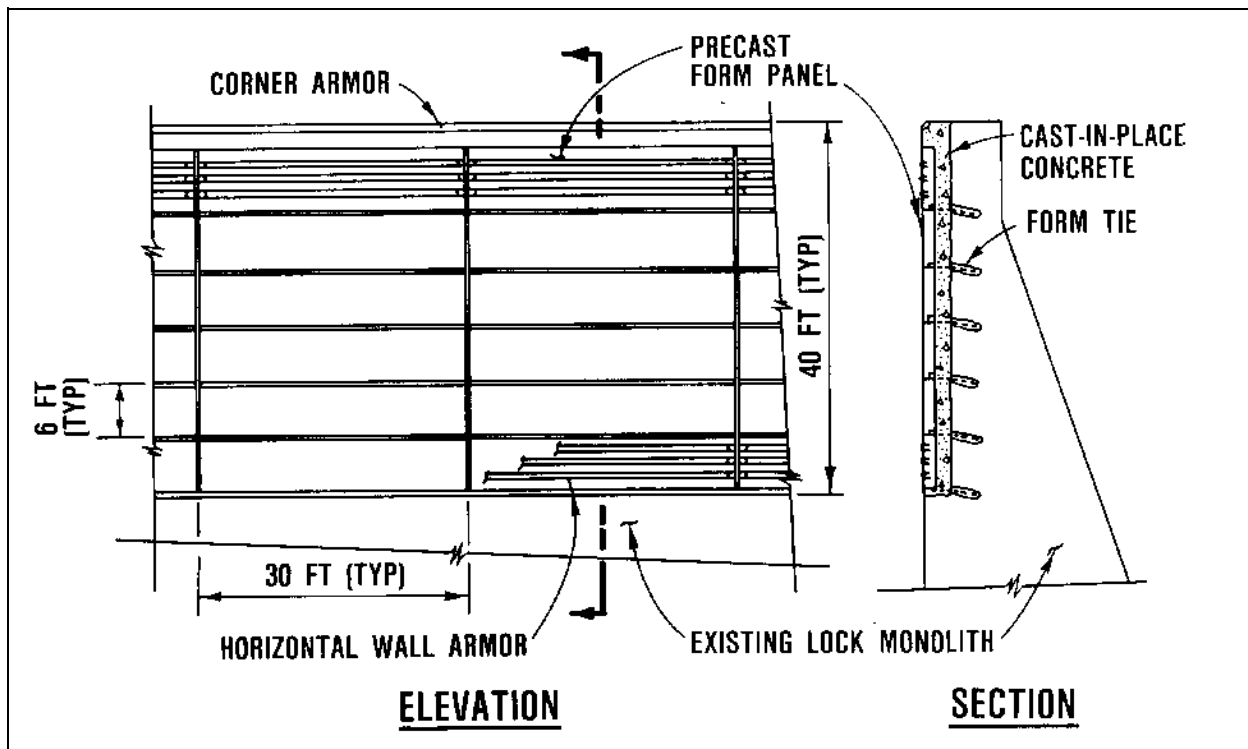


Figure 8-12. Precast concrete stay-in-place forming system for lock wall rehabilitation

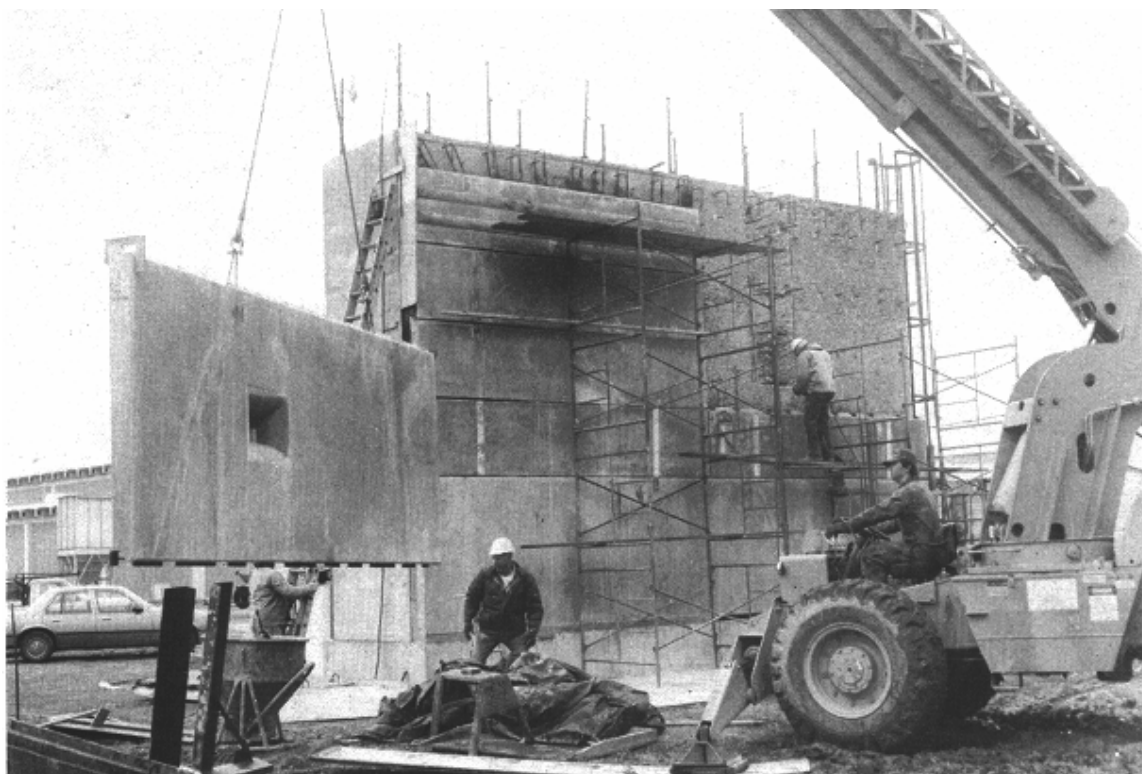


Figure 8-13. Demonstrating the constructibility of the precast concrete stay-in-place forming system

(a) The initial application of the precast-concrete stay-in-place forming system for lock chamber resurfacing was at Lock 22, Mississippi River, near Hannibal, MO, during Jan and Feb 1989. A total of 41 panels were precast in a Davenport, IA, plant with a concrete mixture with an average compressive strength of 54 MPa (7,800 psi). The reinforced-concrete panels were 165 mm (6-1/2 in.) thick, 3.2 m (10-1/2 ft) high, and varied in length from 5.8 to 10.7 m (19 to 35 ft). Weld plates at the top and bottom and leveling inserts along the bottom were embedded in each panel during precasting.

(b) Concrete removal began with a horizontal saw cut at the bottom of the repairs. Line drilling, which was accomplished prior to lock closure, and blasting techniques were used to remove a minimum of 216 mm (8-1/2 in.) of concrete from the face of the lock walls. Following final cleanup of the exposed concrete surface, anchors of weldable-grade reinforcing steel were installed to coincide with weld plates in the bottom of the panels.

(c) The panels were transported to the site by river barges and lifted into position with a barge-mounted crane (Figure 8-14). The panels were held on the sawcut ledge while they were leveled with the vertical alignment hardware and the bottom panel anchors were welded to the weld plates embedded in the panels. The top panel anchors were then installed and welded to the embedded weld plates. Each panel was aligned and welded in 2 to 3 hr. The space between the panels and lock wall was filled with nonshrink cementitious grout. Approximately 1,070 sq m (11,500 sq ft) of the lock walls were resurfaced. The bid price for the precast panel repair, including concrete removal, was \$980 per sq m (\$91 per sq ft). Bid prices for this initial application of the precast concrete stay-in-place forming system for lock wall rehabilitation were somewhat higher than anticipated. However, the rehabilitation went very smoothly, despite severe winter weather conditions, and the precast panels were installed in about one-half the time that would have been required for cast-in-place concrete.

(d) This application of the precast concrete stay-in-place forming system demonstrated the potential for eliminating the need to close and dewater a lock during rehabilitation. Consequently, concepts for installation of the system in an operational lock were developed as part of the REMR research program (ABAM 1989). A mobile cofferdam was selected as the preferred installation method and a final design completed for this concept. This study indicated that it is feasible to repair the walls in an operational lock with only minimal impact on costs.

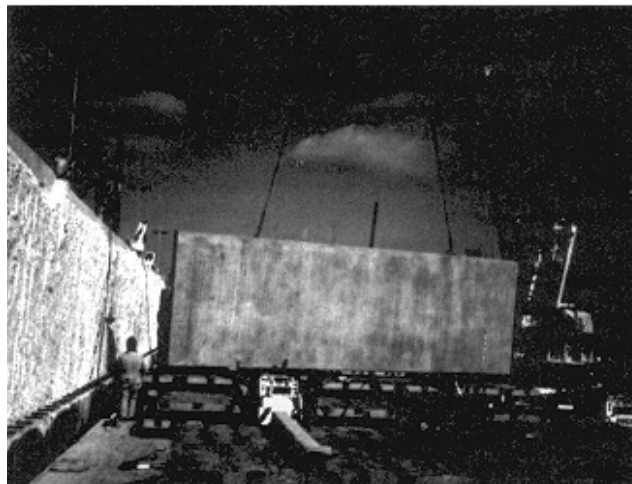
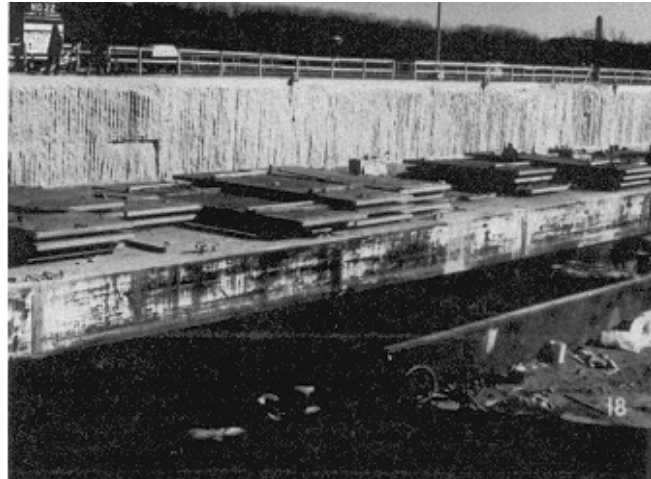


Figure 8-14. Installation of precast concrete stay-in-place forms at Lock 22, Mississippi River

lock wall repairs, they are potentially applicable to other walls requiring underwater repair.

(e) After the lock was reopened to navigation, the concrete along vertical monolith joints began to exhibit cracking and spalling caused by barge impact and abrasion. Recessing the joints by cutting the concrete with a diamond saw eliminated this problem. The cuts were started at a 5-deg angle to the chamber face and 305 mm (12 in.) outside of the joints to produce a 25-mm (1-in.)-deep recess at the joints. Cracked concrete along the joints was removed with a chipping hammer, and the spalled areas were repaired with a latex-modified cement dry pack.

(f) Based on the experience gained at Lock 22, a number of revisions were incorporated into the design of the precast panel system used at Troy Lock, Hudson River, during the winter of 1991-92. These revisions included new lifting, alignment, anchorage, and joint details (Miles 1993). Also, the design was based on an allowable crack width of 0.15 mm (0.006 in.) compared to 0.25 mm (0.01 in.) at Lock 22.

(g) A total of 112 precast concrete panels were required to resurface the lock chamber walls. A typical panel was 190 mm (7-1/2 in.) thick, 3.6 m (11 ft 10 in.) high, and 6.1 m (20 ft) long. Ten special panels were required to accommodate line poles and ladders. These panels were 521 mm (1 ft 8-1/2 in.) thick except at the recesses where the thickness was reduced to 190 mm (7-1/2 in.). The panels were fabricated with a 25 by 305 mm (1 by 12 in.) taper and a 25 mm (1 in.) chamfer along the vertical joints to reduce impact spalling. Reinforcing mats with 51 mm (2 in.) of concrete cover were placed at both faces of each panel. Panel anchors were hoop-shaped reinforcing bars which extended 102 mm (4 in.) from the rear face of the panel. Two erection anchors were embedded in the top of each panel. A concrete mixture proportioned with a 0.31 water-cementitious ratio for a compressive strength of 48 MPa (7,000 psi) at 28 days was used to precast the panels. The panels were fabricated in a heated building during the winter of 1991-92.

(h) A minimum of 305 mm (12 in.) of concrete was removed from the lock walls by saw cutting around the perimeter of the removal area, line drilling, and explosive blasting. Following air-jet cleaning of the surface, anchors were grouted into holes drilled into the existing concrete. These bent reinforcing bar anchors were located to line up with the hoop anchors embedded in the precast panels. The majority of the panels were installed during the winter of 1991-92 when ambient temperatures were generally cold, at times reaching -18 °C (0 °F). The precast panels were lifted from delivery trucks with a crane and lowered onto steel shims placed on the sawcut ledge. The panels were positioned with temporary holding and adjustment brackets, shebolt form anchors at the top and bottom of the panels at 1.2-m (4-ft) spacings, and steel strongbacks spanning between the shebolts. Once the panels were positioned and aligned, vertical reinforcing bars were manually installed from the top down to intersect the panel and wall anchors (Figure 8-15). Once the bars were properly located, they were tied to the anchors to prevent misalignment during placement of the infill concrete.

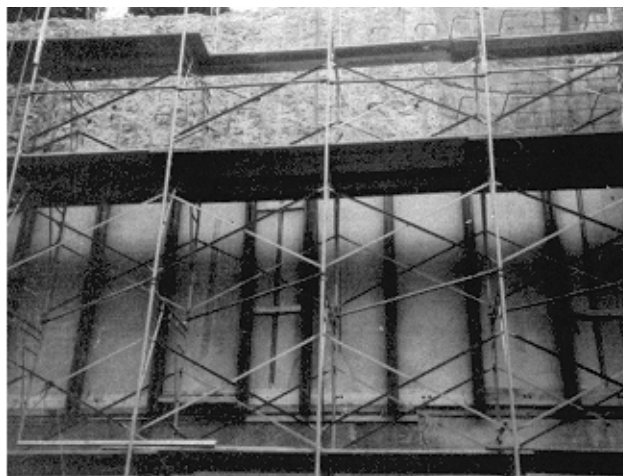


Figure 8-15. Installation of precast concrete stay-in-place forms at Troy Lock, Hudson River (Miles 1993)

(i) The top of the precast panels was capped with 0.6 m (2 ft) of reinforced concrete, cast-in-place. A 152-mm (6-in.)-thick apron slab was also placed on top of the existing monolith wall to reduce water penetration and improve durability. Joints between panels and concrete cap and between cap and apron slab were sealed with joint sealant. Upon completion of the lock chamber resurfacing, an inspection of the precast panels revealed that 11 of 112 panels exhibited some fine cracks. Those cracks (four locations) with widths in excess of 0.15 mm (0.006 in.) were repaired by epoxy injection. In contrast, the miter gate monoliths previously resurfaced with cast-in-place concrete exhibited extensive cracking (Figure 8-16).

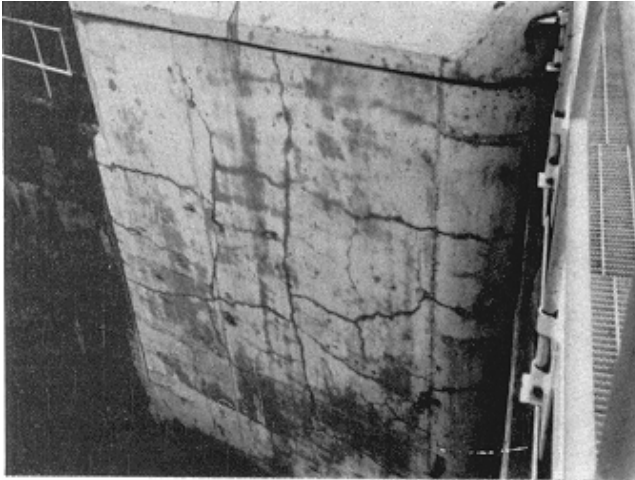


Figure 8-16. Typical cracking in miter gate monoliths resurfaced with cast-in-place concrete, Troy Lock, Hudson River (Miles 1993)

(j) The contractor's bid price for the precast concrete resurfacing in the lock chamber was only \$355 per sq m (\$33 per sq ft) at Troy Lock compared to \$980 per sq m (\$91 per sq ft) at Lock 22. Also, the mean bid price for precast concrete at Troy Lock was approximately \$50 per sq m (\$5 per sq ft) lower than the mean bid price forcast-in-place concrete during the same period. Although the contractor at Troy was inexperienced in both lock rehabilitation and the use of precast concrete, the project progressed quite smoothly and the efficiencies of using precast concrete became very obvious as the work was completed. It is anticipated that as the number of qualified precasters increases and as contractors become more familiar with the advantages of precast concrete, the costs of the precast concrete stay-in-place forming system will be reduced.

(k) In addition to resurfacing the lock chamber, precast concrete panels were used to overlay the back side of the river wall at Troy Lock (Figure 8-17). Original plans for repair of this area required extensive removal of deteriorated concrete and replacement with shotcrete. This repair, which would have had to be accomplished in the dry, would have required construction of an expensive cofferdam to dewater the area. Therefore, it was decided that concrete removal could be minimized and the need for a cofferdam eliminated if this area was repaired with precast concrete panels. Three rows of precast panels were used in the overlay. The panels were installed in 1992 while the lock was in operation. The bottom row of panels was partially submerged during installation and



Figure 8-17. Installing precast concrete panels used to overlay the back side of the river wall at Troy Lock, Hudson River (Miles 1993)

infill concrete placement. An antiwashout admixture allowed the infill concrete to be effectively placed underwater without a tremie seal having to be maintained. The application of precast concrete in this repair resulted in an estimated savings of approximately \$500,000 compared to the original repair method. Also, the durability of the aesthetically pleasing precast concrete should be far superior to shotcrete which has a generally poor performance record in repair of lock walls.

(l) Precast concrete panels were also used to overlay the lower, battered section of the backside of the river wall at Lockport Lock, Illinois Waterway (Figure 8-18). Before the panels were placed, the concrete surface was cleaned of all loose concrete with a high-pressure water jet. Then the precast concrete base was positioned and anchored along the horizontal surface at the base of the wall. In addition to conventional reinforcement in each direction, the precast panels were prestressed vertically. Embedded items included weld plates at the bottom of the panels and steel angles with embedded bolts at the top of the panels. Typical panels were 152 mm (6 in.) thick, 7.1 m (23.2 ft) high, and 2.3 m (7.5 ft) wide.

(m) The panels were installed with a crane. Steel shims were placed between the panel and the base at the weld plates. At the top of the panel, steel angles aligned with the steel angles embedded in the panel were secured to the wall with bolts. After the bolted connections were tightened at the top of the panel, the steel shims at the bottom of the panel were welded to weld plates in the

sq m (\$21 per sq ft), are currently performing satisfactorily.

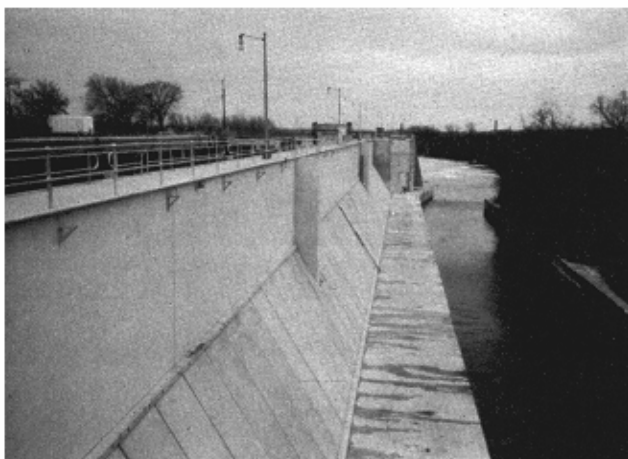
f. Summary. In the design of lock wall repairs, the depth of frost penetration in the area should be considered, particularly in repair of old nonair-entrained concrete. It appears that if the thickness of any repair section is less than the depth of frost penetration, freeze-thaw deterioration of the existing nonair-entrained concrete should be expected. This deterioration can drastically affect the performance of repair sections without adequate anchoring systems. Also, the cost of alternative repairs should be carefully evaluated in relation to the desired service life of the rehabilitated structure. Only a few years of good service, at best, should be expected of shotcrete in relatively thin layers. However, the cost of such a repair will be relatively low. In comparison, conventionally formed and placed concrete, shotcrete, and preplaced-aggregate concrete, each properly proportioned and placed in thicknesses greater than the depth of frost penetration, should provide a minimum of 25 years of service but at successively greater initial costs. Precast concrete panels used as stay-in-place forms should provide even greater durability at approximately the same cost as cast-in-place concrete.

8-2. Repair of Waterstop Failures

Nearly every concrete structure has joints that must be sealed to ensure its integrity and serviceability. This is particularly true for monolith joints in hydraulic structures such as concrete dams and navigation locks. Embedded waterstops are used to prevent water passage through the monolith joints of such structures. Traditionally, waterstops have been subdivided into two classes: rigid and flexible. Most rigid waterstops are metallic: steel, copper, and occasionally lead. A variety of materials are suitable for use as flexible waterstops; however, polyvinyl chloride (PVC) is probably the most widely used (EM 1110-2-2102). Waterstops must be capable of accommodating movement parallel to the axis of the waterstop as a result of joint opening and closing caused by thermal expansion and contraction. Also, differing foundation conditions between adjacent monoliths may result in relative movements between monoliths perpendicular to the plane of the waterstop. A review of waterstop failures (McDonald 1986) reported leakage through monolith joints ranging from minor flows to more than 2,270 L/min (600 gal/min). In general, leakage was the result of (1) excessive movement of the joint which ruptures the waterstop, (2) honeycombed concrete areas adjacent to the waterstop, (3) contamination of the waterstop surface which prevents



a. Installation of precast concrete panels



b. River wall complete with precast concrete panels

Figure 8-18. Backside of river wall, Lockport Lock, Illinois Waterway

base and the panel. The precast headcap sections were lifted into place, set on shims between the top of the panel and the cap, and anchored to the wall. Backer rods and joint sealant were used to seal all joints. The 1,073 sq m (11,550 sq ft) of precast panels, which were installed in the fall of 1989 for a bid price of \$226 per

bond to the concrete, (4) puncture of the waterstop or complete omission during construction, and, (5) breaks in the waterstop due to poor or no splices. Since it is usually impossible to replace an embedded waterstop, grouting or installation of secondary waterstops is the remedial measure most often used. Several types of secondary waterstops have been tried with various degrees of success and expense. McDonald (1986) identified more than 80 different materials and techniques that have been used, individually and in various combinations, to repair waterstop failures. Some repairs appear to have been successful, while many have failed. Generally, repairs can be grouped into four basic types: surface plates, caulked joints, drill holes filled with elastic material, and chemically grouted joints. The particular method used depends on a number of factors including joint width and degree of movement, hydraulic pressure and rate of water flow through the joint, environment, type of structure, economics, available construction time, and access to the upstream joint face.

a. Surface plates. A plate-type surface waterstop (Figure 8-19) consists of a rigid plate, generally stainless steel, spanning the joint with a neoprene or deformable rubber backing. The plate is attached with anchors which provide an initial pressure on the deformable pad. Water pressure against the plate provides additional pressure so that the deeper the waterstop is in a reservoir, the tighter the plate presses to the surface of the joint. This type waterstop has had varying degrees of success. At John Day Lock, it has worked relatively well but at Lower Monumental Lock, it has not performed satisfactorily. The major difference between these structures is that at Lower Monumental, the joint movement is more than that at John Day. Potential problems with this type of repair include: (1) loosening of the anchor bolts; (2) reverse hydrostatic pressure from water trapped behind the waterstop when the reservoir drops; (3) mechanical abuse such as a barge tearing off the plate; (4) ice pressure from

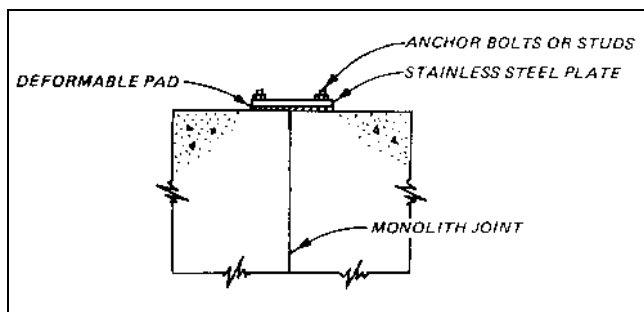


Figure 8-19. Typical plate type waterstop

moisture trapped behind the plate; and (5) hardening of the flexible pad due to aging (Schrader 1980).

b. Caulked joints. When the reservoir level can be dropped below the elevation to which repairs are necessary, and if the joint opening is not too great, a simple and economical repair may be possible by saw cutting along the joint on the positive pressure, or reservoir side, and then filling the cut with an elastic sealant (Figure 8-20). The sawcut should be wide enough to span the joint and cut about 3 mm (1/8 in.) minimum into the concrete on each side of it. The cut must also be deep enough to penetrate any unsound, cracked, or deteriorated materials. Typically, a cut 13 mm (1/2 in.) wide by 38 mm (1-1/2 in.) deep is acceptable. The cut should follow the joint to its base; otherwise, water migrating underneath the sealant can build up pressure in the joint behind the sealant. To perform satisfactorily, the sealant must set rapidly, bond to cool and damp concrete, and remain flexible under anticipated service conditions, and it must not sag in vertical applications or extrude through the joint under hydrostatic pressure. Once problems of partial extrusion due to inadequate curing were solved, this type of repair exhibited reasonable success for several years at Lower Monumental and Little Goose Locks.

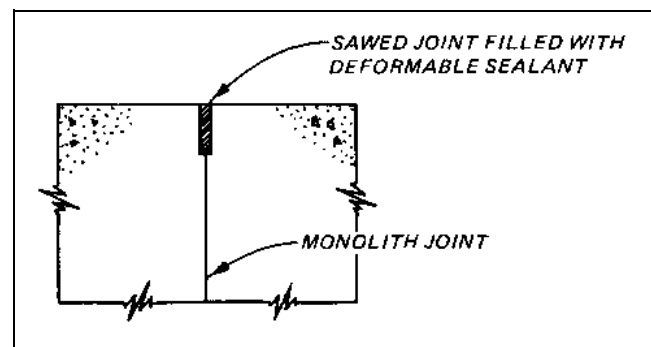


Figure 8-20. Typical sawed joint filled with sealant

c. Drill holes filled with elastic material. This approach (Figure 8-21) consists of drilling a large-diameter hole from the top of the monolith along the joint and into the foundation, and filling the hole with an elastic material. Typically, the hole is 76 to 152 mm (3 to 6 in.) in diameter and drilled by a "down-the-hole" hammer or core drill. The more costly core drilling allows easy visual inspection of hole alignment along the joint, but the down-the-hole hammer has proven to be the preferred method of drilling. Small underwater video cameras can also be used to inspect hole alignment and condition of the joint.

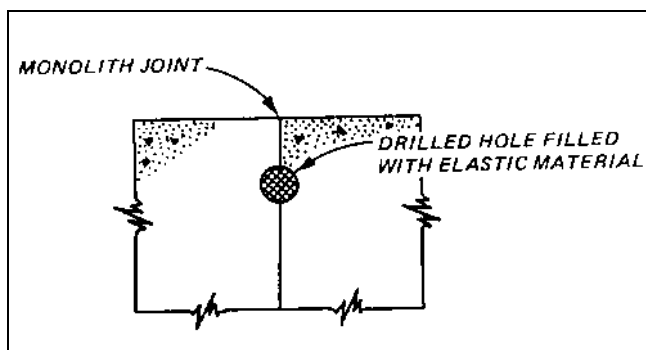


Figure 8-21. Drilled hole with elastic filler

(1) Elastic filler material. The drill holes are filled with an elastic filler material to establish a seal against water penetration through the joint. Criteria for the filler require that it displace water, attain some degree of bond to the concrete surface, remain elastic throughout the life of the structure, be practical for field application, be economical, and have sufficient consistency not to extrude under the hydraulic head to which it will be subjected. If the filler material which is often in liquid form travels out from the drilled hole during placement and into the joint before it sets, better sealing can be expected. The design assumption, however, is that the “poured-in-place” grout filler will form a continuous elastic bulb within the drilled hole. The filler will press tightly to the downstream side of the hole when water pressure is applied to the upstream side, thereby creating a tight seal. Various types of portland-cement and chemical grout have been used as a filler.

(a) Acrylamide grout systems have been used as elastic filler in several applications. Developed primarily for filling voids in permeable sand and gravel foundations, these systems consist of an acrylamide powder, a catalyst, and an initiator. When dissolved in separate water solutions and mixed together, a gelatinous mass results. The reaction time can vary from a few seconds to as much as an hour based on the proportions of the mixture and the temperature. As long as the reacted mass remains in a moist environment, it will stay stable in size and composition and will remain highly flexible. If allowed to dry, its volume can decrease by as much as 90 percent. If resaturated with water, it will regain most of its original properties. The mass can be given more stability, weight, and rigidity through the addition of inert mineral fillers such as diatomaceous earth, bentonite, and pozzolans. In some early applications, portland cement was used to thicken the grout and allow its use in open flowing holes with substantial hydraulic head. However, as the cement hydrated, the grout mass hardened resulting in a

nonflexible filler. Repairs of waterstops using acrylamide grout systems have performed with varying degrees of success. Repairs at Ice Harbor and Lower Monumental Locks, which have been subjected to small relative joint movements, have performed well and are presently functioning as designed. Similar repairs at Little Goose Lock, subject to large relative joint movements, have not performed well. Large movements and high pressures caused the repair material to extrude from the joint in a matter of months. Similar repairs at Pine Flat Dam initially stopped leakage through the repaired joints; however, when the reservoir pool was raised, subjecting the joints to a 61-m (200-ft) head of water, the repairs failed. The potential for extrusion of filler material in unlined drill holes subject to high heads should be recognized.

(b) Water-activated urethane foam grout systems have also been used as elastic fillers. These hydraulic polymers are activated when placed in contact with water and upon curing form a tough, flexible gel approximately 10 times its original volume. One approach in using these materials is to stuff burlap bags which have been saturated with the grout into the drill holes (Figure 8-22). Water in the drill holes activates the grout system causing the grout to expand and completely fill the holes. Tamping the bags as they are pushed into the holes with drill stem will force some of the grout into the joints prior to gel formation. Gel times will vary with ambient and water temperatures but generally can be controlled with additives to be in the range of 5 to 10 min. This procedure was used at Dardanelle Lock with a reduction in gallery leakage of approximately 95 percent. Technical representatives for the material manufacturers should be contacted for detailed guidance on specific grout systems.

(2) Liner materials. In some cases, a continuous tube-type liner has been inserted into the drill hole (Figure 8-23) to contain the filler material. Liner materials include reinforced plastic firehose, natural rubber, elastomer coated fabric, neoprene, synthetic rubber, and felt tubes. In most cases, the liners are not bonded to the walls at the drill holes and rely on differential pressure between the interior of the liner and the external water level to force the liner against the sides of the hole. Obviously, any type of liner material should be of sufficient flexibility to allow the tube to conform to any voids and irregularities in the surface of the drill hole and to accommodate differential movement between adjacent monoliths.

(a) A repair procedure which was developed based on techniques used for in situ relining of pipelines, allows



a. Saturating burlap bags with chemical grout



b. Saturated bag on top of drill hole awaiting insertion

Figure 8-22. Remedial waterstop installation, Dardanelle Lock

bonding of the liner to the surface of the drill hole. In this procedure, the liner is fabricated from thin polyurethane film with an under layer of felt approximately 6 mm (1/4 in.) thick. Prior to installation, water-activated polyurethane resin is poured into the tube to saturate the felt lining. The resin is distributed evenly throughout the tube by passing it through a set of pinch rollers. The tube is then inserted into the drill hole, and as it is inserted water pressure is used to turn the tube inside-out (Figure 8-24). When the inversion process is complete, the resin-impregnated felt is in contact with the surface of



Figure 8-23. Two-component natural rubber and elastomer coated nylon sleeve being inserted into drill hole, Bagnell Dam

the drill hole (Figure 8-25). Water inside the drill hole activates the resin grout creating a bond between the liner and the drill hole. This procedure was used to repair three leaking joints at Pine Flat Dam in March 1985. The drill holes ranged in depth from 58 to 78 m (189 to 257 ft) with a combined flow of 515 L/min (136 gal/min). After repair, the flow dropped to 45 L/min (12 gal/min). As a result of the excellent performance of these repairs, the same procedure was used in 1993 to correct leakage through additional monolith joints at Pine Flat Dam.

(b) A variety of materials have been used as fillers inside the drill hole liners. These include water, bentonite slurry, and various formulations of chemical grout. Criteria for the filler are generally the same as those for drill holes without liners. Filler grouts should have a density greater than that of water and should be placed by tremie tubes starting at the bottom of the liner to displace all water in the liner. Once the filler material is in place, the hole should be capped flush with the top surface. The cap should contain a removable plug for periodic inspection of the filler material.

(c) Video inspections of the drill holes before and after insertion of the liner have proven to be beneficial in determining the exact location of the concrete-foundation interface, irregularities in the concrete surface, seepage locations, and adequacy of the insertion process. Also, if the inversion technique is proposed for liner insertion, the bottom 25 percent of the drill hole should be

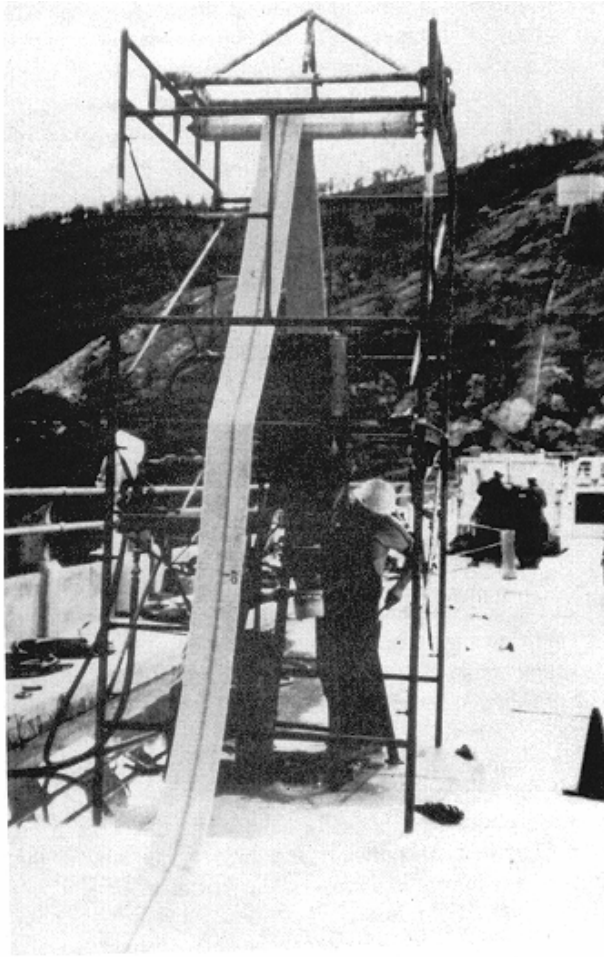


Figure 8-24. Inversion apparatus for installing tube in drill hole, Pine Flat Dam

pressure-tested to determine if leakage rates in this zone are sufficient to allow water in the hole to escape during inversion.

d. Chemically grouted joints. Chemical grouting has been successfully used to seal isolated areas of interior leakage such as around the perimeter of a drainage gallery where it crosses a joint and contraction joints in regulating outlet conduits. Grouting has also been used to seal exterior joints such as those on the upstream face of a dam. Chemically grouted smooth joints often fail when subjected to small relative movement.

(1) Interior joints. The repair procedure consists of drilling an array of small-diameter holes from various locations within the gallery or conduit to intercept the

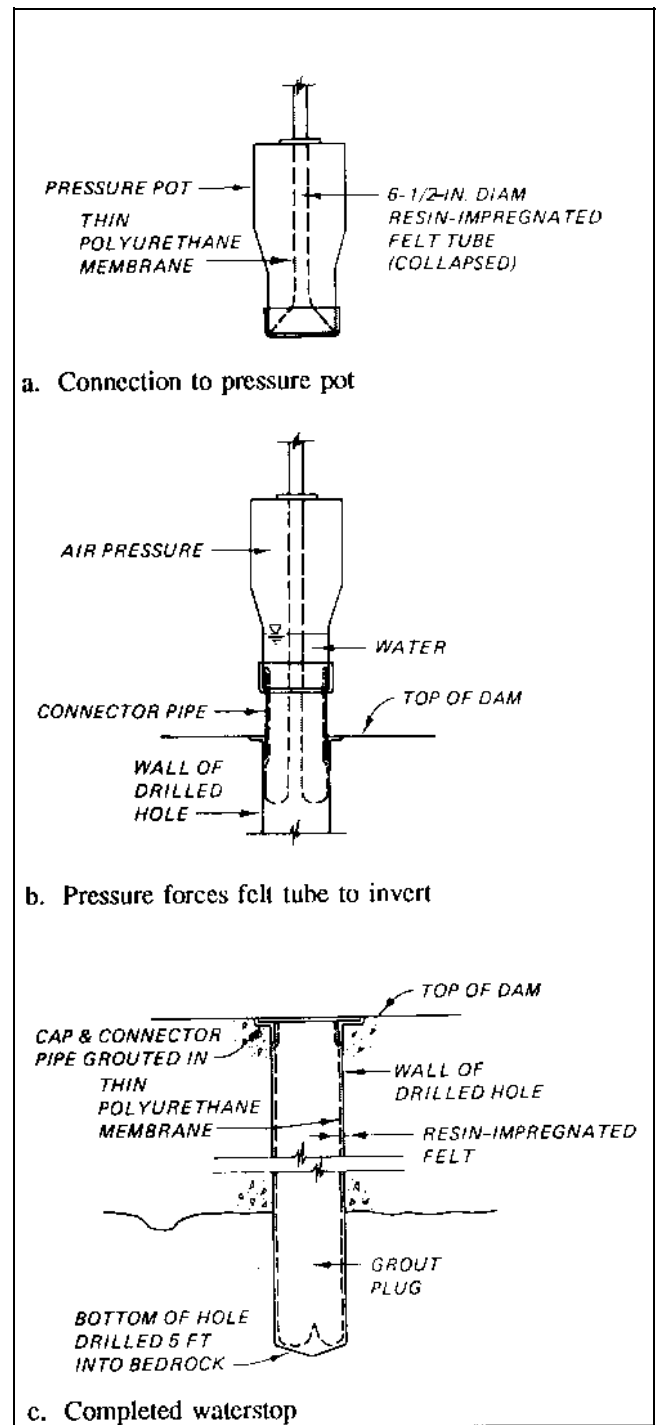


Figure 8-25. Remedial waterstop installation, Pine Flat Dam

joint behind the waterstop (Figure 8-26). An elastic chemical grout is then injected through the drill hole. Criteria for the injection grout require that it have low

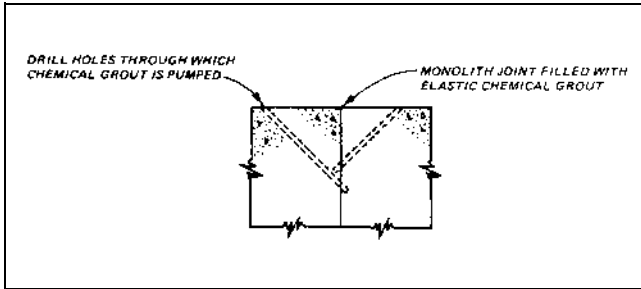


Figure 8-26. Joint filled with elastic grout through injection

viscosity, gel or set quickly, bond to wet surfaces, be suitable for underwater injection, possess good elastic strength, and tolerate unavoidable debris. Water-activated polyurethane injection resins generally meet the desired criteria. Gel times for these materials are normally in the range of 5 to 60 sec which is usually adequate for low-flow conditions. Large-volume or high-pressure flows must be controlled during grout injection and curing. Materials and methods commonly used to control such flows include lead wool hammered into the joint, foam rubber, or strips of other absorbent materials soaked in water-activated polyurethane and packed into the joint, oakum packed into the joint, and small-diameter pipes embedded in the packing material to relieve pressure and divert flow. If the joint opening is greater than approximately 2.5 mm (0.1 in.), a surface plate waterstop may be necessary within the gallery or conduit to prevent grout extrusion with time because of hydraulic pressure along the joint.

(2) Exterior joints. A combination of grouted joint and surface plate waterstop techniques was used to seal vertical contraction joints on the upstream face of the Richard B. Russell Dam (Figure 8-27). A permeable grout tube was placed in the vertical vee along the face of each joint and covered with an elastomeric sealant. After

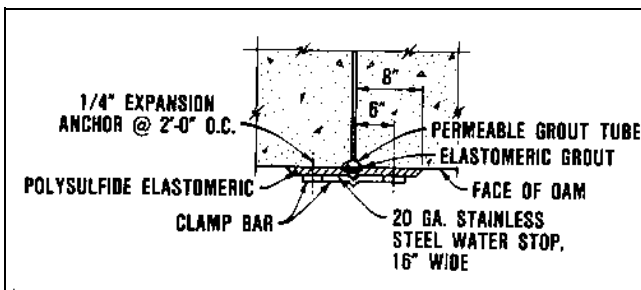


Figure 8-27. Combination grouted joint and surface plate-type waterstop, Richard B. Russell Dam

the sealant hardened, the grout tube was injected with a polysulfide sealant to fill the joints from the dam face into the embedded waterstop. The polysulfide sealant was also placed on the face of the dam for a distance of 203 mm (8 in.) on either side of the joints. Prior to hardening of the sealant, a surface plate waterstop of 20-gauge stainless steel was anchored into position over the joint.

e. Summary. Because of a lack of appropriate test methods and equipment, most of the materials and procedures described have been used in prototype repairs with limited or no laboratory evaluation of their effectiveness in the particular application. Consequently, a test apparatus was designed and constructed, as part of the REMR research program, to allow systematic evaluation of waterstop repair techniques prior to application in prototype structures. The apparatus consists of two concrete blocks, one fixed and one movable, with a simulated monolith joint between the blocks (Figure 8-28). The performance of waterstop repairs can be evaluated for water heads up to 76 m (250 ft) and joint movements up to 10 mm (0.4 in.). Preliminary results of current short-duration tests indicate that grouted joint repairs and most caulked joint repairs begin to leak at joint movements between 1.3 and 2.5 mm (0.05 and 0.1 in.). Catastrophic failure, generally caused by debonding of the repair material, usually occurs at a joint movement of 2.5 mm (0.1 in.) for polyurethane grouts and 5 to 10 mm (0.2 to 0.4 in.) for most joint sealants. To date, the most successful repair consisted of polyurethane grout injection of the joint combined with caulking of the upstream face. This repair exhibited no leakage for joint movements up to 5 mm (0.2 in.) and a water head of 70 m (230 ft).

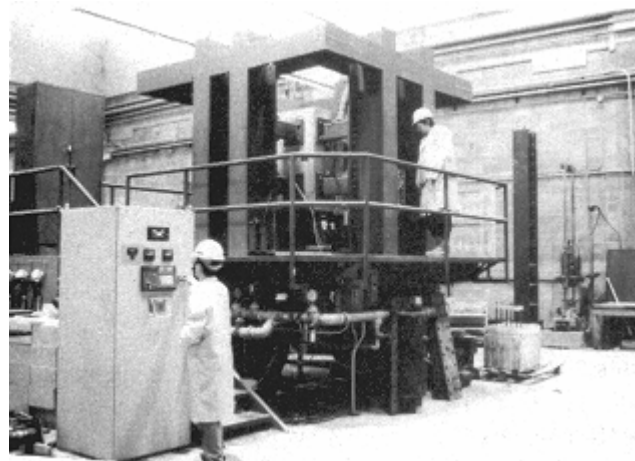


Figure 8-28. Remedial waterstop test apparatus

Leakage was first observed at a joint movement of 8 mm (0.3 in.) and a water head of 38 m (125 ft); however, the magnitude of the leakage was too small to measure for water heads up to 76 m (250 ft). Additional repair materials and procedures are currently being evaluated and a REMR report will be published upon completion of these tests. In the meantime, available test results can be obtained by contacting U.S. Army Engineer Waterways Experiment Station, Structures Laboratory, Concrete Technology Division, (CEWES-SC).

8-3. Stilling Basin Repairs

A typical stilling basin design includes a downstream end sill from 0.9 to 6 m (3 to 20 ft) high to create a permanent pool for energy dissipation of high-velocity flows. Unfortunately, these pools also serve in many cases to trap rocks, reinforcing steel, and similar debris. In most cases the presence of debris and subsequent erosion damage are the result of one or more of the following: (a) construction diversion flow through constricted portions of the stilling basin; (b) eddy currents created by diversion flows or powerhouse discharges adjacent to the basin; (c) construction activities in the vicinity of the basin, particularly those involving cofferdams; (d) nonsymmetrical discharges into the basin; (e) flow separation and eddy action within the basin to transport riprap from the exit channel into the basin, and (f) topography of the outflow channel (McDonald 1980). While high-quality concrete is capable of resisting high water velocities for many years with little or no damage, the concrete cannot withstand the abrasive grinding action of entrapped debris. In such cases, abrasion-erosion damage ranging in depth from a few inches to several feet can result, depending on the extent of debris and the flow conditions. A variety of repair materials and techniques including armored concrete, conventional concrete, epoxy resins, fiber-reinforced concrete, polymer-impregnated concrete, preplaced-aggregate concrete, silica-fume concrete, and tremie concrete have been used to repair erosion damage in stilling basins. Applications of the various repair materials are described in detail in the 31 case histories reported by McDonald (1980). Selected case histories are summarized in the following.

a. Old River Control Structure. Prefabricated modules of steel plate anchored to the top of the end sill and to the floor slab directly behind the downstream row of baffles were used in repair of the stilling basin at the Old River Control Structure. Thirty modules, 7.3 m (24 ft) long and varying in widths from 0.9 to 6.7 m (3 to 22 ft), were fabricated from 13-mm (1/2-in.)-thick steel plate. Vertical diaphragm plates were welded to the

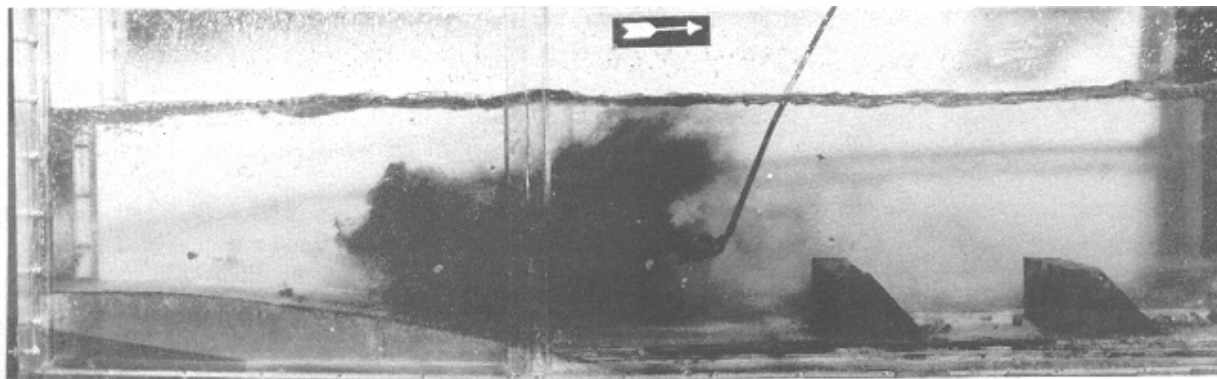
horizontal plate, both to stiffen the plate and to provide a formed void in which to retain the grout. Individual modules were positioned and anchored underwater using polyester resin grouted anchors. A portland-cement grout containing steel fibers was then pumped into the modules.

(1) An underwater inspection 8 months after the repairs showed 7 of the 30 modules had lost portions of their steel plate ranging from 20 to 100 percent of the surface area. A number of anchor bolts were found broken either flush with the plate, flush with the grout, or pulled completely out. In those areas where steel plating was lost, the exposed grout surfaces showed no evidence of significant erosion.

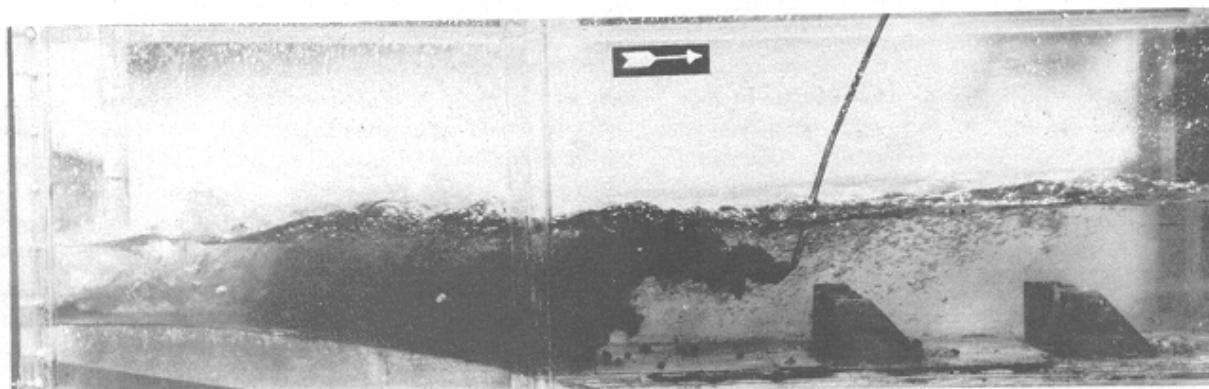
(2) A second inspection, approximately 2 years after the repair, revealed that additional steel plating had been ripped from four of the modules previously damaged. Also, an additional nine modules had sustained damage. Minor erosion had occurred in the stilling basin slab upstream from the modules. The stilling basin was reported to be free from rock and other debris. Apparently, any rock or debris discharged through the structure was flushed from the stilling basin over the fillet formed by the modules at the end sill.

(3) Subsequent underwater inspections indicated continuing loss of the steel plate until the stilling basin was dewatered for inspection and repair in 1987, 11 years after the original repair. Following removal of silt which covered most of the stilling basin floor, an inspection indicated that approximately 90 percent of the steel plate was missing. Protruding portions of the remaining plate were cut flush with the surface, and the leading edge of the grout wedge was removed to a minimum depth of 305 mm (12 in.). Isolated areas of erosion damage and spalling in the grout fillet were repaired with shotcrete. The remainder of the basin received a 305-mm (12-in.)-thick overlay of portland-cement concrete (0.45 w/c).

b. Pomona Dam. Prior to repair, a hydraulic model study of the existing stilling basin at Pomona Dam was conducted to investigate discharge conditions which might account for debris in the basin and to evaluate potential modifications to eliminate these conditions. Model tests (Oswalt 1971) confirmed that severe separation of flow from one side wall and eddy action strong enough to circulate stone in the model occurred within the basin for discharges and tailwaters common to the site. Photographs (Figure 8-29) of subsurface upstream flow in the right side of the basin show the results of a discharge rate of 28 cu m/sec (1,000 cu ft/sec). Based on the model



a. Tailwater elevation A



b. Tailwater elevation B

Figure 8-29. Subsurface upstream flow in the stilling basin at Pomona Dam (after Oswalt 1971)

study, it was recommended that the most practical solution was to provide a 0.9-m (3-ft)-thick overlay of the basin slab upstream of the first row of baffles; a 0.5-m (1.5-ft) overlay between the two rows of baffles; and a 1:1 sloped-face to the existing end sill. This solution provided a wearing surface for the area of greatest erosion and a depression at the downstream end of the basin for trapping debris. However, flow separation and eddy action were not eliminated by this modification. Therefore, it was recommended that a fairly large discharge sufficient to create a good hydraulic jump without eddy action be released periodically to flush debris from the basin.

(1) The final design for the repair included (a) a minimum 13-mm (1/2-in.)-thick epoxy mortar applied to approximately one-half of the transition slab; (b) an epoxy mortar applied to the upstream face of the right three upstream baffles; (c) a 0.6-m (2-ft)-thick concrete overlay

slab placed on 70 percent of the upstream basin slab; and (d) a sloped concrete end sill. The reinforced-concrete overlay was recessed into the original transition slab and anchored to the original basin slab. The coarse aggregate used in the repair concrete was Iron Mountain trap rock, an abrasion-resistant aggregate. The average compressive strength of the repair concrete was 47 MPa (6,790 psi) at 28 days.

(2) The stilling basin was dewatered for inspection 5 years after repair. The depression at the downstream end of the overlay slab appeared to have functioned as desired. Most of the debris, approximately 0.8 cu m (1 cu yd) of rocks, was found in the trap adjacent to the overlay slab. The concrete overlay had suffered only minor damage with general erosion averaging about 3 mm (1/8 in.) deep with maximum depths of 13 mm (1/2 in.). The location of the erosion coincided with that occurring

prior to the repair. Apparently, debris was still being circulated at some discharge rate.

(3) The epoxy mortar overlay had not suffered any visible erosion damage; however, cracks were observed in several areas. In one of these areas the epoxy mortar coating was not bonded to the concrete. Upon removal of the mortar in this area (approximately 2 sq m (25 sq ft)), it was observed that the majority of the failure plane occurred in the concrete at depths up to 19 mm (3/4 in.). Following removal, the area was cleaned and backfilled with a low modulus, moisture-insensitive epoxy mortar. In all other areas, even those with cracks, the epoxy mortar overlay appeared to be well bonded.

(4) Based on a comparison of discharge rates and slab erosion, before and after the repair, it was concluded that the repair had definitely reduced the rate of erosion. The debris trap and the abrasion-resistant concrete were considered significant factors in this reduction.

(5) The next inspection, 5 years later, indicated that the stilling basin floor remained in good condition with essentially no damage since the previous inspection. Approximately 4 cu m (5 cu yd) of debris, mostly rocks, was removed from the debris trap at the downstream end of the basin (Figure 8-30).

c. *Nolin Lake Dam.* The conduit and stilling basin at Nolin were dewatered for inspection in 1974. Erosion was reported in the lower portion of the parabolic section, the stilling basin floor, the lower part of the baffles, and along the top of the end sill. The most severe erosion was in the area between the wall baffles and the end sill where holes 0.6 to 0.9 m (2 to 3 ft) deep had been eroded into the stilling basin floor along the sidewalls (Figure 8-31).

(1) Dewatering of the stilling basin for repair was initiated in May 1975. The structural repair work included raising the stilling basin floor elevation 228 mm (9 in.) and raising the end sill elevation 0.3 m (1 ft). Nonreinforced conventional concrete designed for 34-MPa (5,000-psi) compressive strength was used in the repair. A hydraulic model study of the existing basin was not conducted; however, the structure was modified in an attempt to minimize entry of debris into the basin. New work included adding end walls at the end of the stilling basin and a 15-m (50-ft)-long concrete paved channel section (Figure 8-32). Also, a concrete pad was constructed adjacent to the right stilling basin wall to permit a mobile crane to place a closure at the end of the stilling basin wall for more expeditious dewatering of the basin.

(2) A diver inspection of the stilling basin in 1976 indicated approximately 3,600 kg (4 tons) of rock was in the stilling basin. The majority of this rock was from 19 to 127 mm (3/4 to 5 in.) in diameter with some scattered rock up to 305 mm (12 in.) in diameter. The rock, piled up to 0.38 m (1.25 ft) deep, apparently entered the basin from downstream. Also, piles up to 0.46 m (1.5 ft) deep of similar rock were found on the slab downstream from the stilling basin. Erosion up to 203 mm (8 in.) deep was reported for concrete surfaces which were sufficiently cleared of debris to be inspected.

(3) A similar inspection in August 1977 indicated that approximately 900 to 1,350 kg (1 to 1-1/2 tons) of large, limestone rock, all with angular edges, was scattered around in the stilling basin. No small or rounded rock was found. Since the basin had been cleaned during the previous inspection, this rock was first thought to have been thrown into the basin by visitors. When the stilling basin was dewatered for inspection in October 1977, no rock or debris was found inside the basin. Apparently, the large rock discovered in the August inspection had been flushed from the basin during the lake drawdown when the discharge reached a maximum of 208 cu m/sec (7,340 cu ft/sec). No additional damage had occurred in the stilling basin since the 1976 inspection.

(4) Significant erosion damage was reported when the stilling basin was dewatered for inspection in 1984. The most severe erosion was located behind the wall baffles (Figure 8-33) similar to that prior to repair in 1975. Each scour hole contained well-rounded debris. Temporary repairs included removal of debris from the scour holes and filling them with conventional concrete. Also, the half baffles attached to each wall of the stilling basin were removed. A hydraulic model of the stilling basin was constructed to investigate potential modifications to the basin to minimize chances of debris entering the basin with subsequent erosion damage to the concrete.

(5) Results of the model study were incorporated into a permanent repair in 1987. Modifications included rebuilding the ogee section in the shape of a "whales back;" overlaying the basin floor; adding a sloping face to the end sill; raising the basin walls 0.6 m (2 ft); paving an additional 30 m (100 ft) of the retreat channel; slush grouting derrick stone in the retreat channel; and adding new slush grouted riprap beside the basin.

(6) The condition of the concrete was described as good with no significant defects when the basin was dewatered for inspection in August 1988. The maximum

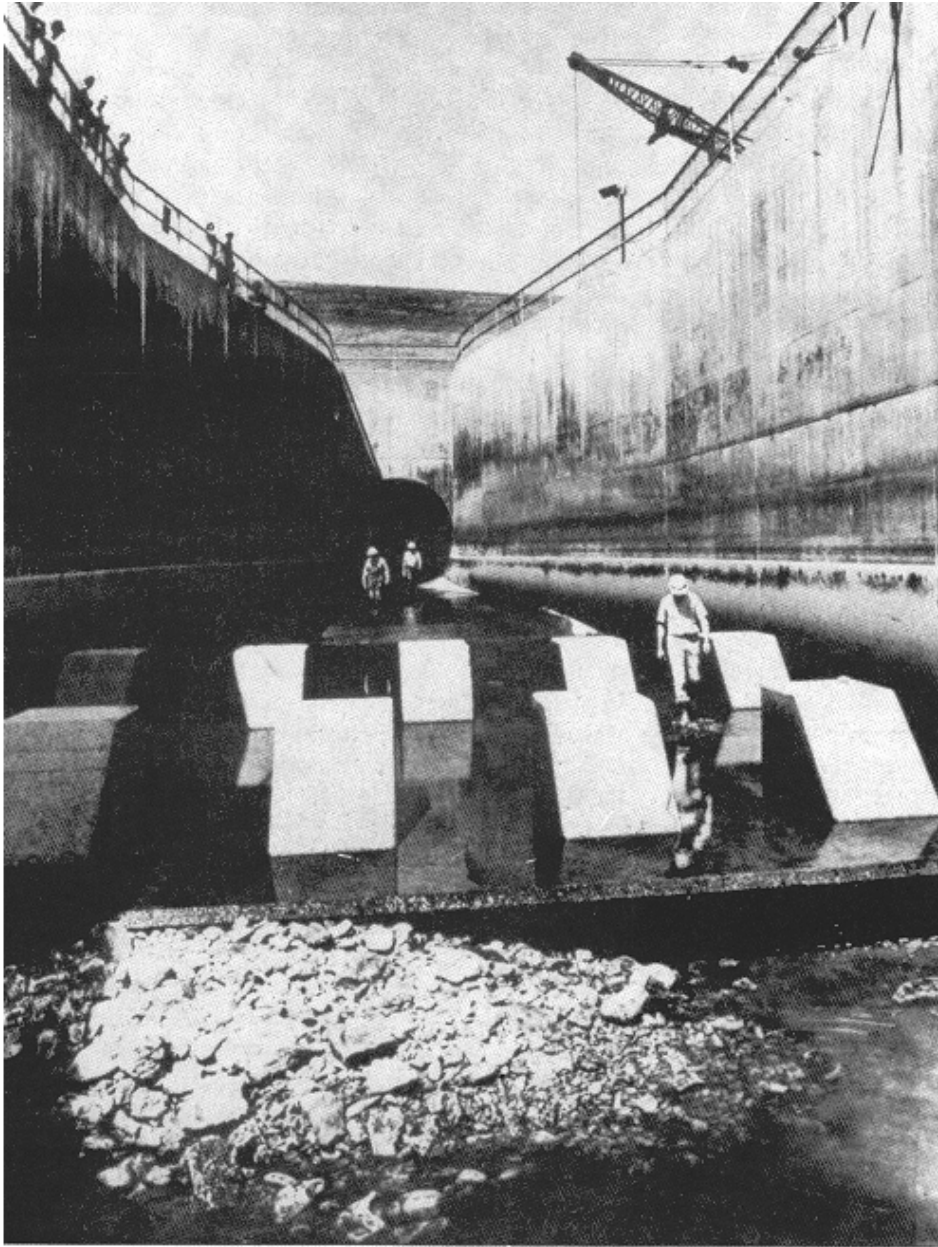


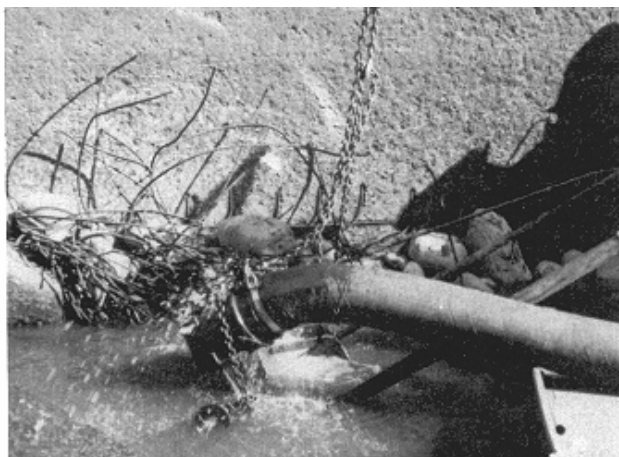
Figure 8-30. Stilling basin dewatered for inspection, Pomona Dam

discharge to that point had been 143 cu m/sec (5,050 cu ft/sec) for a period of 13 days.

d. Kinzua Dam. Because of the proximity of a pumped storage power plant on the left abutment and problems from spray, especially during the winter months, the right-side sluices at Kinzua Dam were used most of the time. This mode of operation caused eddy currents which carried debris into the stilling basin. The fact that

the end sill was below streambed level contributed to the deposition of debris in the basin. As a result, erosion of the concrete to depths of 1 m (3.5 ft) was reported less than 4 years after the basin was placed into normal operation.

(1) The initial repair work (1973-74) was accomplished in two stages; cellular cofferdams enclosed about 60 percent of the stilling basin for each stage, permitting



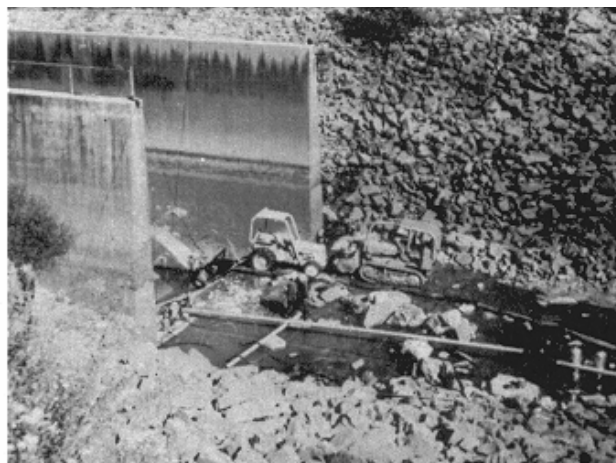
a. Debris collected behind wall baffle



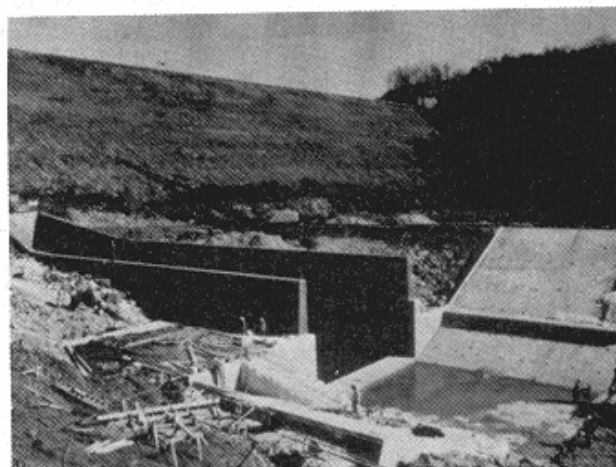
b. Erosion behind wall baffle

Figure 8-31. Concrete erosion damage, Nolin Dam stilling basin, 1974

stream flow in the unobstructed part of the stilling basin. Approximately 1,070 cu m (1,400 cu yd) of fiber-reinforced concrete was required to overlay the basin floor (Figure 8-34). A concrete mixture containing 25-mm (1-in.) steel fibers, proportioned for 8 and 41 MPa (1,100 and 6,000 psi) flexural and compressive strengths, respectively, was used for the anchored and bonded overlay. The overlay was placed to an elevation 0.3 m (1 ft) higher than the original floor from the toe of the dam to a point near the baffles.



a. Prior to repair



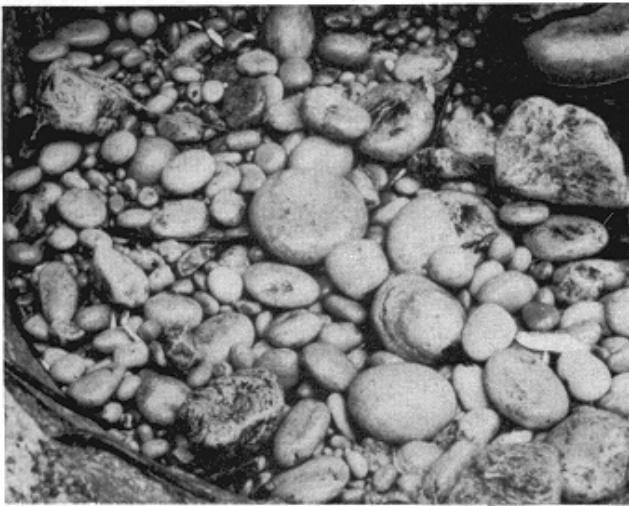
b. After repair

Figure 8-32. Modifications to outlet work, Nolin Dam, 1975

(2) The initial diver inspection of the repair in November 1974, 1 year after completion of Stage I repairs, indicated minor concrete deterioration in some areas of the basin floor. An estimated 34 cu m (45 cu yd) of debris was removed from the basin. In an effort to verify the source of this material, approximately 5,500 bricks of three different types were placed in the river downstream of the basin. Six days later, the basin was inspected by divers who found numerous smooth



a. Scour holes behind baffles



b. Debris in scour holes

Figure 8-33. Concrete erosion damage, Nolin Dam stilling basin, 1984

pieces of all three types of brick in various sizes. This finding proved that the debris was being brought into the stilling basin from areas downstream of the end sill.

(3) In April 1975, additional abrasion-erosion damage to the fiber-reinforced concrete was reported. Maximum depths ranged from 127 to 432 mm (5 to 17 in.). Approximately 34 cu m (45 cu yd) of debris was removed from the stilling basin. Additional erosion was reported in May 1975, and approximately 46 cu m (60 cu yd) of debris was removed from the basin. At this point,

symmetrical operation of the lower sluices was initiated to minimize eddy currents that were continuing to bring large amounts of downstream debris into the stilling basin. The opening of any one gate was not allowed to deviate from that of the other gates by more than 0.3 m (1 ft) initially. This was later reduced to 152 mm (6 in). After this change, the amount of debris removed each year from the basin was drastically reduced, and the rate of abrasion declined; however, nearly 10 years after the repair, the erosion damage had progressed to the same degree that existed prior to the repair.

(4) A materials investigation was initiated prior to the second repair to evaluate the abrasion-erosion resistance of potential repair materials (Holland 1983, 1986). Test results indicated that the erosion resistance of conventional concrete containing a locally available limestone aggregate was not acceptable. However, concrete containing this same aggregate with the addition of silica fume and an HRWRA exhibited high-compressive strengths and very good abrasion-erosion resistance (Figure 8-35). The poor performance of fiber-reinforced concrete cores taken from the original overlay correlates well with the poor performance of the material in the actual structure.

(5) A hydraulic model study of the stilling basin was also conducted prior to the second repair to evaluate potential modifications to the structure that would minimize entry of debris into the basin (Fenwick 1989). Modifications evaluated included a floating boom over the end sill, a downstream dike both longitudinal and lateral, a debris trap, and the paving of the downstream channel. Also, a modification to the upper sluices to provide a manifold to evenly distribute these flows across the stilling basin was investigated. It was suspected that unsymmetrical operation of the lower sluices during the winter months brought material into the basin, while the operation of the upper sluices during the summer months churned the debris around in the basin causing the abrasion-erosion damage. If the debris could not be prevented from entering the basin, then the next approach would be to prevent the swirling discharge from the upper sluices from entering the stilling basin.

(6) Of all the alternatives investigated, it was determined that a debris trap just downstream of the end sill was the least costly with the highest potential for preventing debris from entering the basin. The other alternatives investigated, besides being higher in cost, were subject to possible destruction by spillway flows. The basic purpose behind this plan was to stop the debris from entering the



Figure 8-34. General view of repair operations, Kinzua Dam, 1974

stilling basin by restricting upstream flows and thereby allowing the debris to drop into the 7.6-m (25-ft)-wide trap. The top of the debris trap wall was at the same elevation as the top of the end sill and extended across the stilling basin width. The concrete debris trap wall was an inverted tee wall founded on a varying rock

foundation. The debris trap wall was designed to withstand the hydraulic forces during spillway flows.

(7) It was believed that the symmetrical operation of the lower sluice gates would totally prevent the transportation of material into the stilling basin. Therefore, the

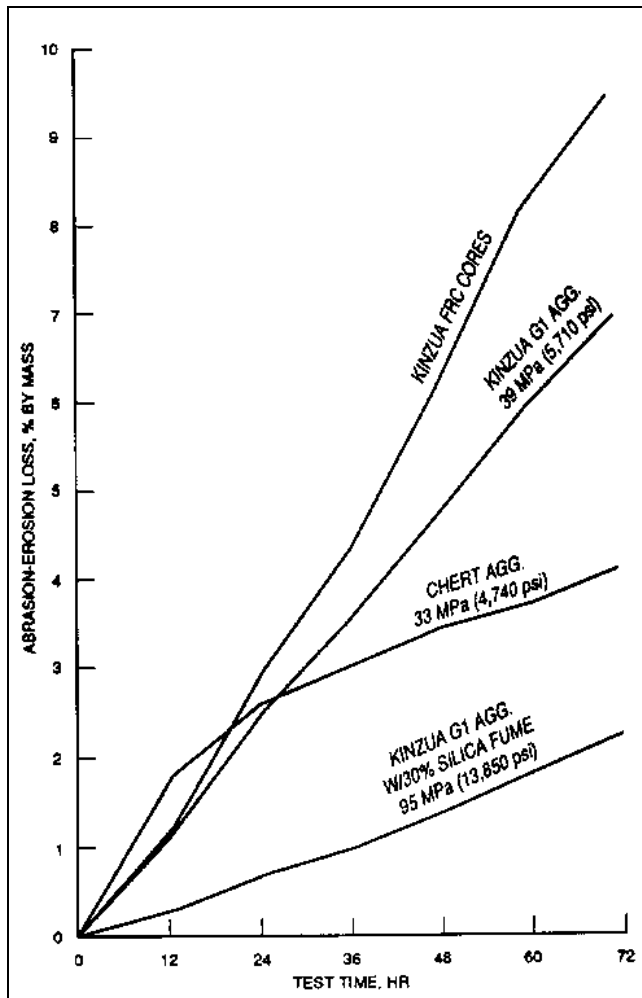


Figure 8-35. Results of selected abrasion-erosion tests, Kinzua Dam

debris trap would serve only as insurance against debris entering the stilling basin for any unanticipated reason. Although prototype studies indicated that velocities were not high enough to bring debris in, the fact remained that debris was removed from the stilling basin annually. Since the operation of the sluice gates could also be controlled by the power company, there was concern that the gates could be accidentally or unintentionally operated in an unsymmetrical manner. Previous studies indicated that unsymmetrical operation for even a few hours would bring in large amounts of debris.

(8) Approximately 1,530 cu m (2,000 cu yd) of silica-fume concrete was used in a 305-mm (12-in.) minimum thickness overlay when the stilling basin was repaired in 1983 (Holland et al. 1986). The bid price for silica-fume

concrete was \$353/cu m (\$270/cu yd). Construction of a debris trap immediately downstream of the stilling basin end sill was also included in the repair contract. The trap was 7.6 m (25 ft) long with a 3-m (10-ft)-high end sill that spanned the entire width of the basin.

(9) Following dewatering and a thorough cleanup, the transit-mixed silica-fume concrete was pumped into the basin through approximately 46 m (150 ft) of 127-mm (5-in.) pump line (Figure 8-36). The concrete was consolidated with internal vibrators. Once the majority of the concrete was in the form, the screeding operation was started. The two vibrating screeds were connected in tandem with the second following the first at about a 1.5-m (5-ft) interval. Curing compound was applied immediately after the second screed passed over the concrete.

(10) Although the placements generally went well, there was a problem that arose during the construction: cracking of the silica-fume concrete overlay. The widths of cracks were from 0.25 to 0.51 mm (0.01 to 0.02 in.) and usually appeared 2 to 3 days after placement. The cracks were attributed primarily to restraint of volume changes resulting from temperature gradients and, possibly, autogenous shrinkage. Several approaches, including applying insulating blankets over the concrete, saw cutting the slab after placing to establish control joints, and addition of reinforcing steel mat, were attempted to stop or minimize the cracking; however, no solution was found for the cracking problem. The cracks were not repaired because laboratory testing of cracked specimens indicated that fine cracks do not affect abrasion resistance.

(11) Concrete materials and mixture proportions similar to those used in the Kinzua Dam repair were later used in laboratory tests to determine those properties of silica-fume concrete which might affect cracking (McDonald 1991). Tests included compressive and tensile splitting strengths, modulus of elasticity, Poisson's ratio, ultimate strain capacity, uniaxial creep, shrinkage, coefficient of thermal expansion, adiabatic temperature rise, and abrasion erosion. None of these material properties, with the possible exception of autogenous shrinkage, indicated that silica-fume concrete should be significantly more susceptible to cracking as a result of restrained contraction than conventional concrete. In fact, some material properties, particularly ultimate strain capacity, would indicate that silica-fume concrete should have a reduced potential for cracking.



Figure 8-36. Pumping silica-fume concrete into the stilling basin from the top of the right training wall, Kinzua Dam

(12) The potential for cracking of restrained concrete overlays, with or without silica fume, should be recognized. Any variations in concrete materials, mixture proportions, and construction practices that will minimize shrinkage or reduce concrete temperature differentials should be considered. Where structural considerations permit, a bond breaker at the interface between the replacement and existing concrete is recommended.

(13) Four diver inspections were made in the year following the placement of the overlay. During the first inspection (April 1984), very little deterioration was noted and the divers estimated that the curing compound was still in place over 90 percent of the slab. The divers recovered about one and one-half buckets of small gravel and a small reinforcing bar from the basin after the inspection. The second diver inspection was made in August 1984, after a period of discharge through the upper sluices. Erosion to about 13 mm (1/2 in.) in depth was found along some of the cracks and joints. The divers also discovered a small amount of debris and two pieces of steel plating that had been embedded in the concrete around the intake of one of the lower sluices. Because of concern about further damage to the intake, the use of this sluice in discharging flows was discontinued. This nonsymmetrical operation of the structure

resulted in the development of eddy currents. Consequently, a third inspection in late August 1984 found approximately 75 cu m (100 cu yd) of debris in the basin. In September 1984, a total of approximately 380 cu m (500 cu yd) of debris was removed from the basin, the debris trap, and the area immediately downstream of the trap. Debris in the basin ranged in size up to more than 305 mm (12 in.) in diameter. Despite these adverse conditions, the silica-fume concrete continued to exhibit excellent abrasion resistance. Erosion along some joints appeared to be wider but remained approximately 13-mm (1/2-in.) deep.

(14) Sluice repairs were completed in late 1984, and symmetrical operation of the structure was resumed. A diver inspection in May 1985 indicated that the condition of the stilling basin was essentially unchanged from the preceding inspection. A very small amount of debris, approximately 0.1 cu m (3 cu ft), was removed from the basin. The debris trap was also reported to be generally clean with only a small amount of debris accumulated in the corners. A diver inspection approximately 3-1/2 years after the repair indicated that the maximum depth of erosion, located along joints and cracks, was about 25 mm (1 in.).

e. Dworshak Dam. A diver inspection in May 1973, about 2 years after the basin became operational, indicated that rubble and materials from construction of the dam had entered the stilling basin and had caused severe damage to the concrete. An inspection in June 1975 indicated that erosion had progressed completely through the 3-m (10-ft)-thick floor slab in some areas. It was estimated that approximately 1,530 cu m (2,000 cu yd) of concrete had been eroded from the stilling basin. This extensive damage was attributed to two factors: (1) large amounts of construction debris deposited in the basin prior to and during initial operation, and (2) unbalanced flow into the basin because of inoperable or faulty gates in the spillway and outlet works.

(1) The stilling basin was dewatered for repair in 1975. An anchored overlay of fiber-reinforced concrete (381-mm (15-in.) minimum thickness) was applied to the basin floor. Flexural and compressive strengths of the fiber concrete mixture were approximately 6 and 55 MPa (860 and 8,000 psi), respectively, at 28 days. Following completion of the overlay, the concrete in the right half of the basin was impregnated with methyl methacrylate monomer (Schrader and Kaden 1976).

(2) In areas where erosion of the original concrete was less than the 381 mm (15-in.) minimum depth specified for the overlay, the design called for removal of the existing concrete to this depth. However, one section of the stilling basin floor and the lower portion of the spillway exhibited only minimal erosion to a maximum depth of about 100 mm (4 in.). Therefore, it was decided to repair both sections, totaling approximately 475 sq m (5,100 sq ft), with an epoxy mortar topping. Several types of epoxy mortar were used to complete this work; the primary one was a stress-relieving material which was slow curing and had a low exotherm. Several problems which were primarily the result of workmanship, weather conditions, and failure to enclose the work area occurred with the epoxy during application. Under the cool conditions that existed, the epoxy mortar probably did not receive full cure before the stilling basin was put back into service.

(3) During the 7 months between completion of repairs and the initial diver inspection, the basin was subjected to a total of 53 days usage (9 days from the spillway gates and 44 days from the regulating outlets). The spillway and outlet gates were operated symmetrically (or very close to it) for all spills. Total flows varied between 59 and 566 cu m/sec (2,100 and 20,000 cu ft/sec), with the majority being on the order of 85 to 283 cu m/sec (3,000 to 10,000 cu ft/sec).

(4) The underwater inspection of the stilling basin by diver identified isolated accumulations of gravel, rebar, and other debris at a number of locations throughout the basin. Since the stilling basin had been completely cleaned following repair and current erosion was not sufficient to expose reinforcement or produce large-size gravel, it was concluded that the material was entering the basin from downstream of the end sill. The inspection indicated no major erosion or damage. The stilling basin walls had a small amount of surface erosion (less than 25 mm (1 in.)). There were several areas at the junction between the floor and wall with erosion up to 76-mm (3-in.) deep. An estimated 25 percent of the surface area of the epoxy mortar had experienced some degree of failure ranging up to 102-mm (4-in.) depths. The fiber-reinforced concrete (both polymerized and non-polymerized) was generally in good condition. In general, the polymer-impregnated side was probably a little better than the nonpolymerized side. There were several areas of erosion in the center of the basin to depths of about 25 mm (1 in.) deep. Joints and open cracks in the entire basin (including the fiber-reinforced concrete) were the most susceptible to damage. Typical joints and open cracks in the fiber concrete had eroded up to about 25-mm (1-in.) deep at the joint and tapered out to the original floor surface within a foot of the joint. Because of the moisture in the joints and cracks during the repair, concrete at joints and cracks was not impregnated.

(5) Four months later (Nov 1976), after some additional usage of the stilling basin, a diver was employed to clean the debris from the basin and provide more information on the condition of the floor. Significant comments resulting from this inspection were that there were large areas of the concrete surface near the center of the basin with grooves 51 to 76 mm (2 to 3 in.) deep. These grooves, in both the polymerized and nonpolymerized fiber-reinforced concrete, were oriented in the direction of flow.

(6) The next diver inspection of the stilling basin in October 1977 indicated that the basin remained clear of debris. Since there had been no spill between inspections, it was concluded that operation of the powerhouse adjacent to the stilling basin would not in itself cause debris to enter the basin. At this point, a hydraulic model study of the stilling basin was initiated.

(7) The 1:50-scale model demonstrated that debris moved upstream into the stilling basin area as a result of flow conditions at the end of the structure when the spillway was in use (U.S. Army Engineer District, Walla Walla 1979). Eddy action at the edges of the outflow

flume moved 16- to 254-mm (5/8- to 10-in.) gravel and cobble near the basin into the runout excavation along the ends of the basin walls only. This material was carried to the upstream edge of the end sill by the roller beneath the flow separating from the sill. Occasionally, the highly turbulent flow lifted from the edge of the sill and the bottom roller swept material into the basin. With time, sizeable amounts of debris accumulated in the basin. The tendency was that the higher the discharge, the greater the movement of material into the basin.

(8) A 6.1-m (20-ft)-high, 1.5-m (5-ft)-thick sill across the basin 3 m (10 ft) upstream from the existing end sill of the same height formed a rock trap that effectively confined debris coming into the basin. Sills of lower heights were investigated but were unsatisfactory. Low walls that extended downstream from the basin walls were also effective in stopping movement of debris into the basin. Sheet-pile walls that were 4 m (13 ft) higher than the downstream channel and extended 15 m (50 ft) downstream, 1.8 m (6 ft) beyond the end of the runout excavation, blocked the movement of debris at the ends of the basin walls. However, a check of prototype site conditions revealed that the area was in rock and therefore unsuited for sheet piles. A second wall-extension plan had low fills 7.6 m (25 ft) long and a minimum of 1.2 m (4 ft) high (Figure 8-37), which might be constructed of tremie concrete. Although lower and shorter, the fills were as effective as the sheet-pile walls in stopping debris movement and confining the bottom roller. The fills were recommended as the best plan for debris control.

(9) Extension of the right training wall was completed in February 1980. The stilling basin was inspected in September 1981 by use of an underwater television camera. No additional damage was observed during the inspection. The stilling basin was generally clean except for a thin skiff of fine material which had settled on the bottom since the spillway was last used in the spring of 1981.

(10) Inspection of the stilling basin was performed by divers with an underwater video camera in October 1983. Several small areas of apparent erosion were identified on the upstream left side of the basin floor. It was difficult to determine whether these areas were, in fact, erosion of the concrete or remnants from previous stilling basin repairs. During past polymerization of concrete surfaces, thin layers of sand adhered to the surface, resulting in a rough texture. This may explain what appeared to be minor erosion damage.

f. Chief Joseph Dam. The stilling basin at Chief Joseph Dam is approximately 280 m (920 ft) wide and 67 m (220 ft) long and is divided into four rows of concrete slabs approximately 20 m (65 ft) wide, 15 m (50 ft) long, and 1.5 m (5 ft) thick. Extensive areas of eroded concrete were discovered in the stilling basin during an underwater inspection in March 1957, 2 years after the project became operative. By 1966, erosion had progressed to maximum depths of approximately 1.8 m (6 ft), with the most severe erosion located in areas between the row of baffles and the end sill. Because of the high cost of dewatering, it was decided to repair the basin slabs underwater using preplaced-aggregate concrete and pumped concrete.

(1) In the preplaced-aggregate concrete operations, concrete buckets containing the coarse aggregate were guided into position and dumped by a diver. Screeds on preset edge forms were then used by the divers for leveling the aggregate to the proper grade before placing the top form panels. Grout pipes were driven the full depth of the coarse aggregate. The grout mixer and pumps were set up on a training wall and grout was first pumped through grout pipes in the deepest area of the placement until a return appeared through the vent pipes surrounding that area. These were plugged with corks when good sanded grout appeared. When grout appeared in the next row of vent pipes, the grout hose was moved to an adjacent pipe and the grout hole plugged. This procedure was continued until the entire form had been pumped. Approximately 61 cu m (80 cu yd) of preplaced-aggregate concrete was required in the repair.

(2) In the pumped concrete operations, concrete was batched at a local supplier and delivered to the site in transit mixers. At the site, the material was placed in a collection hopper on a training wall and delivered from this point through a 254-mm (10-in.) pipe into concrete buckets on a barge for ferrying to the pump sites. From this point the concrete was pumped through a 76-mm (3-in.) hose. The last 0.6 m (2 ft) of this hose was fitted with a metal tube to allow ready insertion of the conduit into the concrete when the surface leveled off. A clamp was located just above the metal tube to provide the required valve action and allow the placement to be controlled by the diver. The concrete mixture produced a workable material that was easily placed and produced a smooth even surface. Surface slopes estimated at 12:1 were achieved. The measured surface area of pumped concrete was 367 sq m (439 sq yd).

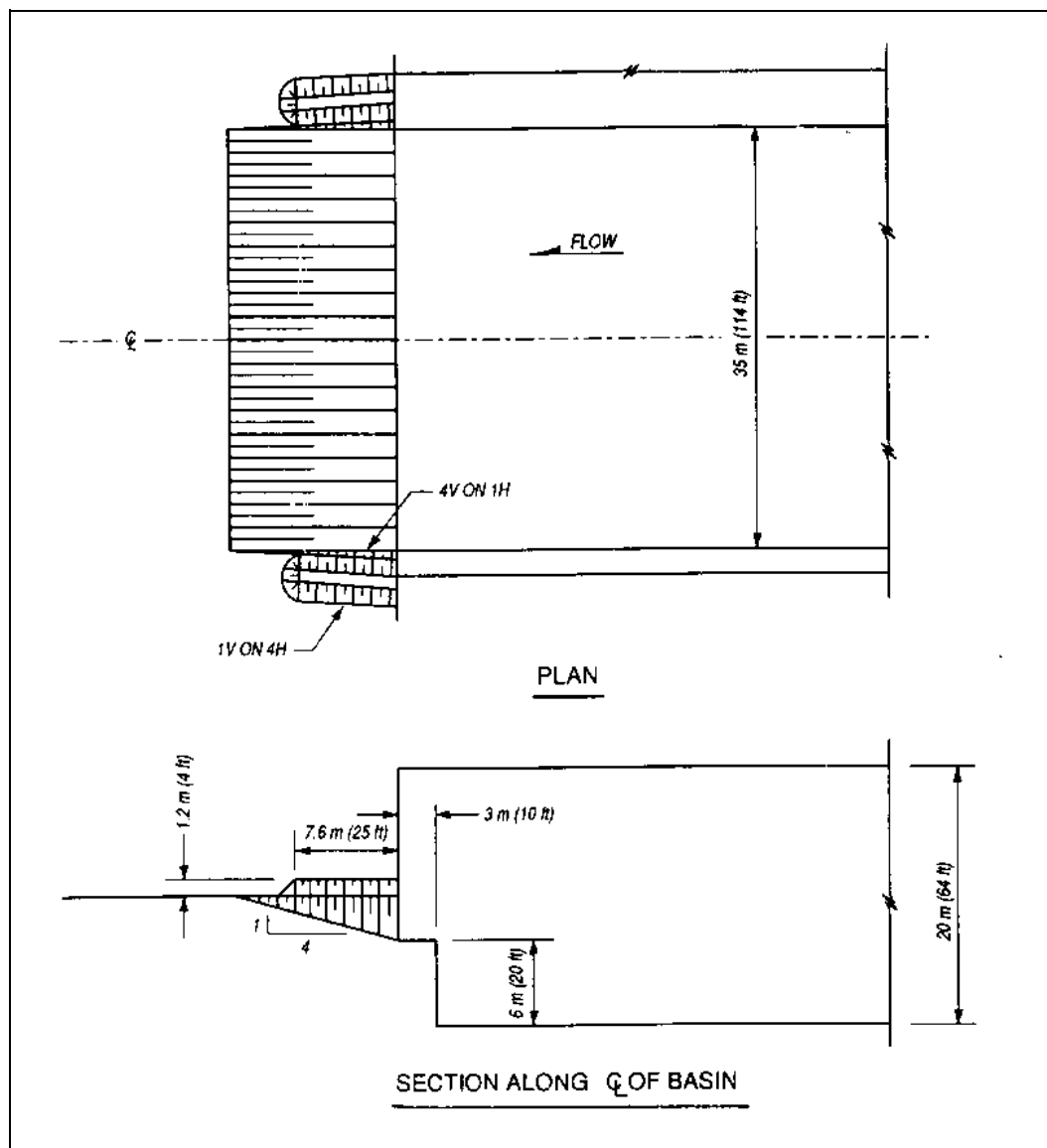


Figure 8-37. Stilling basin wall extension fills, Dworshak Dam

(3) The repairs were accomplished in September through December 1966. During the high-water season in 1967, the repairs were subjected to peak discharges of 12,230 cu m/sec (432,000 cu ft/sec), a flow with a frequency of recurrence of about once in 6 years. A December 1967 inspection of the repairs showed the preplaced-aggregate concrete surfaces to be in excellent condition. The pumped concrete was reported to be in good condition with only minor surface damage noted. The worst damage occurred to the first placement of

pumped concrete where the design mixture was too stiff. A detailed inspection of the stilling basin conducted in 1974 indicated there had been no extensive erosion of the stilling basin since the repairs were made.

g. *Summary.* The repair materials and techniques described in the preceding case histories have been in service for various lengths of time and have been exposed to different operational conditions as well as different levels and durations of flow. This dissimilarity makes

any comparison of the relative merits of the various systems difficult at best; however, a number of general trends are apparent.

(1) Materials.

(a) The resistance of steel plate to abrasion erosion is well established; however, it must be sufficiently anchored to the underlying concrete to resist the uplift forces and vibrations created by flowing water. Welding of anchor systems as nearly flush with the plate surface as possible appears more desirable than raised-bolt connections. In any case, the ability of the anchor system, including any embedment material, to perform satisfactorily under the exposure conditions, particularly creep and fatigue, should be evaluated during design of the repair.

(b) It is feasible to use prefabricated panels or cast-in-place concrete in underwater repair of stilling basins. The inherent advantages of each procedure should be considered in those cases where it is extremely difficult and expensive to dewater a structure to make repairs under dry conditions. Prefabricated elements for underwater repair of stilling basins were investigated as part of the REMR research program (Rail and Haynes 1991). Guidance on underwater repair of concrete is given in Section 8-6.

(c) Minimal erosion of the fiber-reinforced grout at Old River Low Sill Structure has been reported. However, the location of this material on a 1-in-7 slope at the end sill should significantly reduce the potential for the presence of abrasive debris on surface of the repair material. The fiber-reinforced concrete failed under the severe abrasion condition at Kinzua Dam. Also, the poor performance of fiber-reinforced concrete cores in abrasion-erosion tests correlated well with the poor performance of the material in the actual structure. After 1-year exposure to limited discharges at Dworshak Dam, fiber-reinforced concrete (both polymerized and nonpolymerized) was eroded to maximum depths of 51 to 76 mm (2 to 3 in.).

(d) An estimated 25 percent of the epoxy mortar at Dworshak Dam had failed, probably because of workmanship, weather conditions, and lack of sufficient curing during construction. No erosion damage to the epoxy mortar was visible at Pomona Dam; however, there were several areas of cracking and loss of bond attributed to improper curing and thermal incompatibility with existing concrete.

(e) Conventional concrete proportioned for 34-MPa (5,000-psi) compressive strength exhibited erosion up to

203 mm (8 in.) deep after less than 1-year exposure at Nolin Lake Dam. In comparison, conventional concrete containing abrasion-resistant trap rock aggregate had general erosion of only 3 to 13 mm (1/8 to 1/2 in.) after 10 years exposure at Pomona Dam. The abrasion resistance of conventional concrete containing the limestone aggregate locally available at Kinzua Dam was unacceptable. However, concrete containing this same aggregate with the addition of silica fume and an HRWRA has exhibited excellent abrasion-erosion resistance. Unlike concretes containing or impregnated with polymers, concretes containing silica fume are economical and are readily transportable and placeable using conventional methods.

(2) Revised Configuration.

(a) The steel modules anchored to the stilling basin slab and the top of the end sill at Old River Low Sill Structure essentially created a sloping end sill. Apparently, any debris discharged through the structure is being flushed from the basin over this fillet, since diver inspections have reported the basin to be free from debris. When the stilling basin at Pomona Dam was dewatered 5 and 10 years after repair, minimal amounts of debris were found in the stilling basin. However, it is difficult to determine if these relatively small amounts of debris were influenced by the addition of a sloping end sill.

(b) A debris trap was provided in the Pomona Dam stilling basin by eliminating the 0.6-m (2-ft)-thick repair overlay in an area between the downstream baffles and the end sill. The debris trap appears to have functioned as planned because most of the debris found in subsequent inspections was located in the trap adjacent to the raised overlay slab. Strong circulatory currents within the stilling basin appear to have negated any effect of the shallow debris trap incorporated into the original repair at Kinzua Dam. Following the adoption of symmetrical discharges, the small amounts of debris present in the stilling basin were located in areas of erosion throughout the basin. The debris trap with a 3-m (10-ft)-high end sill included in the second repair was also unable to prevent downstream debris from entering the basin under the severe hydraulic conditions at Kinzua Dam.

(c) The stilling basin at Nolin Lake Dam was modified during both repairs. In the second repair, modifications were based on the results of a hydraulic model study. These modifications appear to have eliminated previous erosion problems. The extension of the training wall at Dworshak Dam appears to have enhanced the performance of the stilling basin.

(3) Operations.

(a) Model tests of the stilling basin at Pomona Dam verified that severe separation of flow from one sidewall and eddy action within the basin occurred for discharge and tailwater conditions common to the prototype. Also, these tests revealed that the eddy within the basin was capable of generating considerable reverse flow from the exit channel with the potential to transport riprap from the channel into the basin. Based on the model tests, guidance as to the discharge and tailwater relations required to flush debris from the basin was developed. Following implementation of the most practical material and hydraulic modifications, the stilling basin is performing quite well.

(b) Because of the proximity of a pumped-storage power plant on the left abutment and problems from spray, especially during the winter months, the right-side sluices at Kinzua Dam were used most of the time. This usage caused a circulatory current that carried debris from downstream over the end sill, which is below streambed level, into the stilling basin. Under these conditions, an average of 39 cu m (50 cu yd) of debris was removed from the basin during each of three inspections within the 7 months following completion of repairs. At this point a policy of symmetrical sluice operation was initiated, and based on prototype experiments, a table outlining sluice operating procedure for a range of outflow was prepared. Subsequent to the adoption of this revised sluice operation policy, a minimum of debris was removed from the basin and the rate of erosion decreased.

(c) Upon completion of inspection or repair of a dewatered stilling basin, the basin should be flooded in such a manner to prevent material from temporary access roads, cofferdams, etc., from being washed back into the basin.

h. Conclusions.

(a) Conventional concrete with the lowest practical w/c ratio and hard, abrasion-resistant coarse aggregate is recommended for repair of structures subjected to abrasion-erosion damage. Also, silica-fume concrete appears to be an economical solution to abrasion-erosion problems, particularly in those areas where locally available aggregate otherwise might not be acceptable. The abrasion-erosion resistance of repair materials should be evaluated in accordance with ASTM C 1138 prior to application.

(b) In many cases, underwater repair of stilling basins with prefabricated elements, preplaced-aggregate concrete, pumped concrete, or tremie concrete is an economical alternative to dewatering a stilling basin for repairs under dry conditions. Even when a stilling basin is dewatered, it is often difficult to dry existing concrete surfaces because of leaking cracks and joints. Materials suitable for repair of wet concrete surfaces were identified and evaluated as part of the REMR research program (Best and McDonald 1990b).

(c) Additional improvements in materials should continue to reduce the rate of concrete damage caused by erosion. However, until the adverse hydraulic conditions that caused the original damage are minimized or eliminated, it will be difficult for many of the materials currently being used in repair to perform in the desired manner. Prior to major repairs, model studies of the existing stilling basin and exit channel should be conducted to verify the cause(s) of erosion damage and to evaluate the effectiveness of various modifications in eliminating undesirable hydraulic conditions.

(d) In existing structures, releases should be controlled to avoid discharge conditions where flow separation and eddy action are prevalent. Substantial discharges that can provide a good hydraulic jump without creating eddy action should be released periodically in an attempt to flush debris from the stilling basin. Guidance as to discharge and tailwater relations required for flushing must be developed through model/prototype tests. Periodic inspections should be required to determine the presence of debris in the stilling basin and the extent of erosion.

8-4. Concrete Cutoff Walls

Concrete cutoff walls, sometimes referred to as diaphragm walls, are cast-in-place structures used to provide a positive cutoff of the flow of water under or around a hydraulic structure. The decision to construct a concrete cutoff wall is usually not made as a result of deterioration of the concrete in a structure but rather because of flows under or around the structure. Therefore, the decision to construct such a wall should only be made after a thorough program of geotechnical monitoring and review.

a. Application. Concrete cutoff walls have been used in several instances at structures to reduce or eliminate potentially dangerous flows through foundation materials. The walls are typically unreinforced, approximately 0.6 to 0.9 m (2 to 3 ft) thick and are as deep and as long

as required by site conditions. At Wolf Creek Dam in Kentucky, the wall was 683 m (2,240 ft) long and contained elements as deep as 85 m (278 ft). In some cases, a cutoff wall has been constructed only after attempts at grouting have been unsuccessful or have not given assurances of completely eliminating flows.

b. Procedure. In general, cutoff walls are constructed as summarized in the following steps. Kahl, Kauschinger, and Perry (1991), Holland and Turner (1980), and Xanthakos (1979) provide additional information on construction of concrete cutoff walls.

(1) A concrete-lined guide trench is constructed along the axis of the wall. This trench is usually only a few feet deep. The concrete provides a working surface on both sides of the wall, helps to maintain the alignment of the wall, and prevents the shoulders of the excavation from caving into the trench.

(2) The excavation is accomplished with appropriate equipment for the site conditions. Usually, the excavation is done as a series of discontinuous segments with the project specifications limiting the amount of excavation that can be open at any time. As segments are excavated and backfilled with concrete, intervening segments are constructed. Excavation equipment includes clam shells, rock drills, and specialty bucket excavators. The excavating is usually done through bentonite slurry to keep the holes open. Slurry preparation, handling, and cleaning are critical aspects of the project. Careful control must be exercised during the excavation of each segment to maintain verticality. If a segment is out of vertical alignment, the next adjacent segment to be placed may not contact the first segment for its entire depth and a gap may exist in the wall.

(3) Concrete is placed in the segments using tremies. (See Section 6-33 for a general description of tremie placement.) Since the wall is expected to provide a complete cutoff, any discontinuities in the concrete may cause serious problems in the performance of the wall. Problems that have been reported have included zones or portions of the wall containing poorly or completely uncemented aggregates. These problems are usually attributable to an improperly proportioned concrete mixture or to poor placement practices. It is extremely important that project personnel be familiar with the required procedures for these placements and that specifications be strictly enforced. The concrete mixture itself is very important, as it is for any tremie placement. The specifications for the concrete should not be based upon a required compressive strength. Instead, the flowability and cohesion of

the concrete are critical. Concrete with the proper characteristics may be proportioned using a minimum cement content of 386 to 415 kg/cu m (650 to 700 lb/cu yd) and a maximum w/c of 0.45. Testing as outlined in CRD-C 32 will be of benefit while developing a suitable concrete mixture.

(4) Core drilling should be specified throughout the project as a means of determining the quality of the concrete in place in the wall.

(5) It is an extremely beneficial practice for these projects to require the contractor to place several test panels outside the actual wall area or in noncritical portions of the wall. These test panels will allow for thorough review of the proposed procedures, concrete mixture, and equipment.

8-5. Precast Concrete Applications

The use of precast concrete in repair and replacement of civil works structures has increased significantly in recent years and the trend is expected to continue. Case histories of precast concrete applications in repair or replacement of a wide variety of structures including navigation locks, dams, channels, floodwalls, levees, coastal structures, marine structures, bridges, culverts, tunnels, retaining walls, noise barriers, and highway pavement are described in detail by McDonald and Curtis (1995). Applications of precast concrete in repair of navigation lock walls are described in Section 8-1. Selected case histories of additional precast concrete applications are summarized in the following.

a. Barker Dam. One of the earliest applications of precast concrete panels as stay-in-place forms was at Barker Dam, a cyclopean concrete, gravity structure located near Boulder, CO. The dam is approximately 53 m (175 ft) high with a crest length of 219 m (720 ft). The dam underwent major rehabilitation in 1947 to replace the deteriorated concrete in the upstream face, to correct leakage problems, and to improve the stability of the dam. Deterioration of the concrete on the upstream face of the dam was caused by exposure to approximately 36 years of cycles of freezing and thawing. The reservoir, which is filled in the spring and early summer primarily by melting snow, is drawn down during the winter leaving the upstream face of the dam exposed.

(1) Rehabilitation of the upstream face of the dam consisted of removing the deteriorated concrete, installing precast reinforced-concrete panels over the entire

upstream face, placing coarse aggregate between the dam face and the precast panels, and then grouting the aggregate (Figure 8-38). Several factors influenced the decision to select this repair method, including: (a) repair had to be completed between the time the reservoir was emptied in the fall and filled in the spring, (b) precast panels and aggregate could be placed in severe winter weather conditions, (c) precasting the panels the summer prior to installation and placing them during the winter reduced the potential for later opening of construction joints, (d) a grout with a low cement content could be used for the preplaced aggregate to minimize temperature rise, providing there was a protective shell of high-quality precast concrete, and (e) precast panels were not as expensive as the heavy wooden forms necessary for the placement of conventional concrete.

(2) Resurfacing of the upstream face of the dam required 1,009 precast concrete panels with a total surface area of 7,110 sq m (8,500 sq yd). The reinforced-concrete panels were precast onsite. Each panel was

203 mm (8 in.) thick and most of the panels were 2.1 m (6.75 ft) wide by about 3.7 m (12 ft) long and weighed about 3,630 kg (4 tons). Prior to panel installation, deteriorated concrete was removed from the upstream face of the dam, anchors were installed in the sound concrete, and a stepped footing of conventional concrete was constructed at the base of the dam. The panels were positioned on the dam with a crawler-crane, and dowels embedded in the panel during precasting were welded to anchor bars in the face of the dam. After the joints were grouted, placement of the aggregate was started. Grouting of the preplaced aggregate began when the water elevation was 4.6 m (15 ft) below the crest of the spillway and was completed in about 10 days with almost no interruption.

(3) The panels were erected and coarse aggregate for the preplaced-aggregate concrete was placed concurrently during the period Jan-Apr 1947. Working conditions during this period were generally miserable with bitter

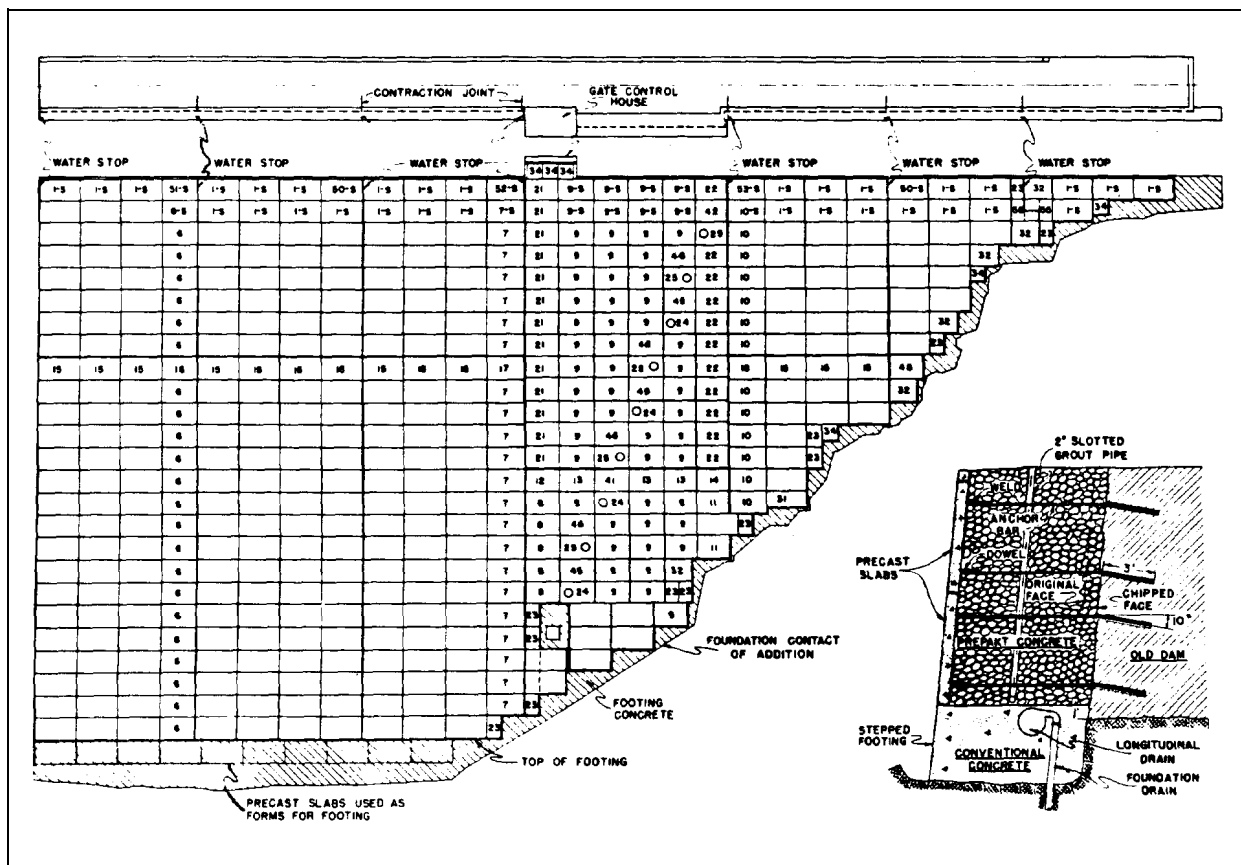


Figure 8-38. Footing and slab layout, south half of Barker Dam (from Davis, Jansen, and Neelands 1948)

cold and high wind velocities. Concrete construction with conventional methods would have been impractical during this period because of the severe weather conditions. The degree of severity of the weather was reflected in rather large daily variations in the rate of panel erection; the average rate was about 12 panels per day with a maximum of 27 panels erected in 1 day.

(4) The quality of the work at Barker Dam is considered to be excellent and the objectives of the rehabilitation program were achieved; however, it is believed that precast panels of much larger size would have resulted in additional economies. With heavy construction equipment, panels up to four times the area could be handled without difficulty and erected at about the same rate as the smaller panels. In addition to reducing the cost of panel erection, larger panels would significantly reduce the total length of joints between panels with a corresponding reduction in the cost of joint treatments.

b. Gavins Point Dam. Concrete spalling along the south retaining wall downstream of the powerhouse at Gavins Point Dam was discovered during a diver inspection. Subsequent inspections, which indicated that the spalling was increasing in area and depth, caused concern that the tailrace slab was being undermined. A repair in the dry would have required construction of a cofferdam; however, the cost of building a cofferdam plus the lengthy powerplant outage during construction was too great. Therefore, it was decided that the repairs would be made underwater by divers, working at a depth of approximately 15 m (50 ft).

(1) After the spalled concrete surfaces were cleaned, the voids were filled with preplaced aggregate, covered by precast concrete panels, and grouted (Figure 8-39). The precast concrete stay-in-place forms were anchored to sound concrete to resist uplift caused by the pressure grouting and to improve stability during power plant operation.

(2) The contract specified that the underwater repair was to be completed during a period of 14 consecutive days when the power units were shut down. The contractor used several diving crews so work could continue 24 hours a day. Also, each step of the repair was reviewed on land before being done underwater. As a result, the project was satisfactorily completed 3-1/2 days early.

c. C-1 Dam. Lock and Dam C-1 is located on the Champlain Canal near Troy, NY. A two-stage rehabilitation of the dam's seven tainter gate piers was initiated in

1993. In stage I, a cellular cofferdam was constructed to enclose piers 5, 6, and 7. Following dewatering, the counterweights for each tainter gate were removed and placed on temporary supports immediately downstream of the gates. The steel tainter gates were then removed for refurbishing and the existing concrete gate piers were removed down to the original foundation. Precast concrete units were used to reconstruct the gate piers (Figure 8-40).

(1) The concrete mixture used in precasting was proportioned with 13-mm (1/2-in.) maximum size aggregate for a 28-day compressive strength of 48 MPa (7,000 psi). The panels were reinforced with Grade 60, epoxy-coated reinforcing steel. The nose and butt units were cast as individual panels and the remaining pier units consisted of three or four integrated panels. Fabrication tolerances for lengths of the units were 13 mm (1/2 in.), plus or minus, or 3 mm (1/8 in.) per 3 m (10 ft) of length, whichever was greater. Panel thickness and unit width tolerances were 6 mm (1/4 in.).

(2) The precast pier units were transported via tractor-trailer rigs to a launch ramp near the dam. At this point, the loaded rigs were driven onto a barge for transportation to the construction site in the river. The pier units were offloaded with a crane located on the cofferdam. A second crane inside the cofferdam was used to position the pier units. After each tier of units was properly proportioned and aligned, the vertical and horizontal joints were grouted, reinforcement was installed inside the units, and the units were filled with conventional concrete. Eighteen precast units were used in each gate pier. The completed piers are 2.4 m (8 ft) wide, 18.7 m (61.5 ft) long, and 8.7 m (28.5 ft) high. Following completion of the gate piers, the reconditioned tainter gates were reinstalled (Figure 8-41). A similar procedure is currently being used to reconstruct the four remaining gate piers.

d. Chauncy Run Checkdams. As part of the overall rehabilitation of the Hornell Local Flood Protection Project, two checkdams were constructed on Chauncy Run. The original plan was to use cast-in-place concrete gravity dams with fully paved stilling basins. However, cost estimates in the 30-percent design submission indicated the use of precast concrete could decrease the cost of the dams by 50 percent. In addition, precasting under controlled plant conditions would assure high quality materials. Precast concrete crib units were used to form the abutments which were then filled with gravel. The checkdams are specially designed precast concrete planks supported by precast concrete posts embedded in the rock

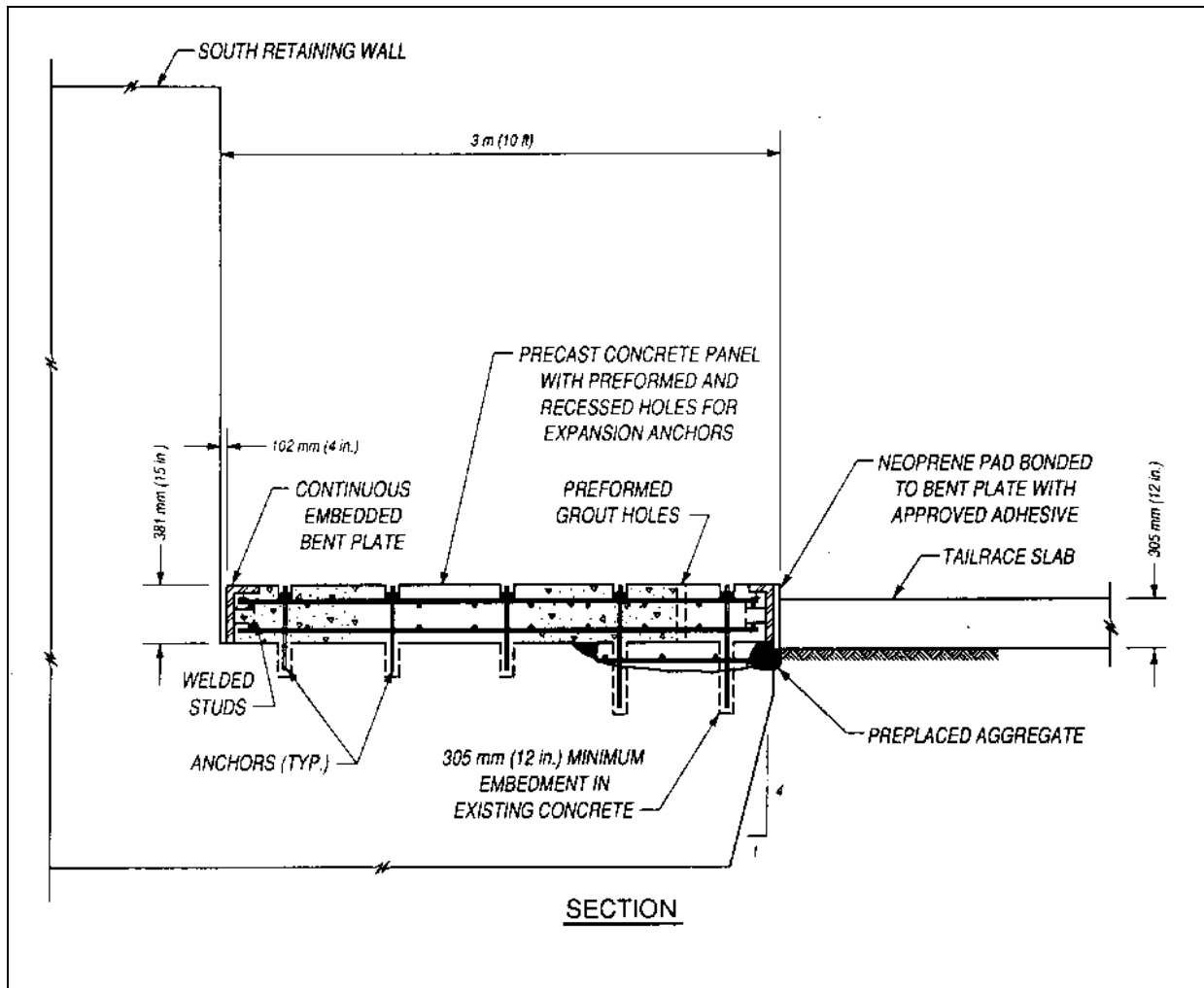


Figure 8-39. Underwater repair of concrete spalling, Gavins Point Dam

foundation (Figure 8-42). The use of precast concrete made it easy to maintain flows during construction, and the appearance of the new structures is well-suited to the site (Figure 8-43).

e. Vischer Ferry Dam. This concrete gravity dam, completed in 1913, is located on the Mohawk River near Albany, NY. The dam consists of two overflow sections with an average height of 12 m (40 ft). The dam was rehabilitated in 1990 as part of an overall project to expand the powerhouse and increase generating capacity. As part of the rehabilitation, the existing river regulating structure was moved to accommodate construction of the expanded powerhouse. The replacement structure is situated perpendicular to the dam so that it discharges from the left side of the new intake. The upstream end of the relocated regulating structure also forms the intake entrance of the forebay. A hydraulic model of the

forebay area showed that head loss and the potential for water separation could be reduced significantly if a contoured pier nose was added at the upstream end of the regulating structure.

(1) The original design for the pier nose was based on cast-in-place concrete inside a dewatered cofferdam. However, the bid cost for the cofferdam alone was \$250,000, so the project team reviewed alternatives and decided to use six precast concrete sections stacked vertically with tremie concrete infill. Placement of the precast nose sections (Figure 8-44) and the tremie concrete required approximately 7 working days. The first section was positioned and leveled with jackposts, and sandbags were placed around the perimeter of this segment. Infill concrete was tremied to the top of this section and cured for 4 days. During this time, divers installed guide angles

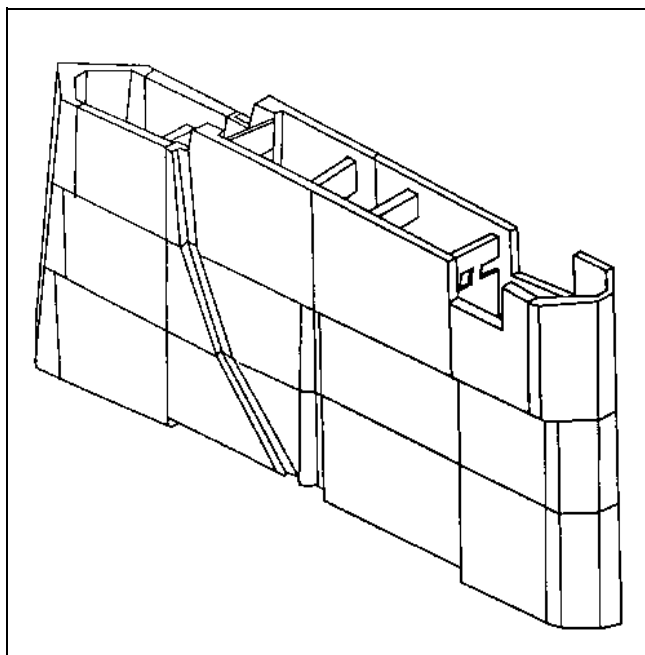


Figure 8-40. Isometric view of gate pier 6, C-1 Dam

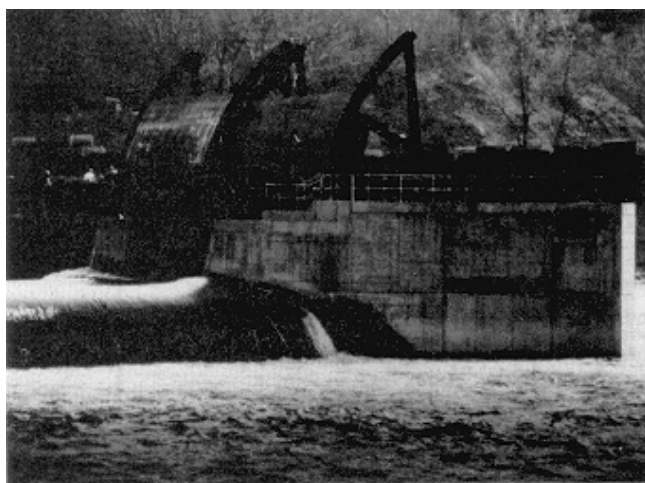


Figure 8-41. Completed precast concrete piers with tainter gates reinstalled, C-1 Dam

and reinforcing dowels into the existing pier and spud pipes into the first section. Then, precast sections 2 through 6 were installed, and temporary intermediate connections were installed to resist plastic concrete loads during the second tremie concrete placement. Tremie concrete was placed to within 152 mm (6 in.) of the surface at a placement rate that did not exceed 3 m (10 ft) per hr. A cast-in-place concrete cap slab was then formed and placed.

(2) Construction of the rounded pier nose with precast concrete and tremie concrete eliminated the need for a cofferdam and resulted in a savings of \$160,000. In addition to reduced construction time and costs, this method effectively eliminated the potentially adverse impact of cofferdam construction on river water quality.

f. Placer Creek Channel. The Placer Creek flood control channel is located in Wallace, ID. For several decades, the channel was repaired and rehabilitated until the channel linings became badly deteriorated and large volumes of debris collected in the channel, reducing its capacity and causing damage to adjacent property.

(1) The contractor elected to use a cast-in-place concrete bottom with precast concrete walls to rehabilitate a 1,128-m (3,700-ft)-long section of the channel (Figure 8-45). Approximately 600 reinforced-concrete panels were precast in a local casting yard. Each panel was 4.6 m (15 ft) long and 3 m (10 ft) high. A 305-mm (12-in.) stub at the bottom of each panel provided continuity of the reinforcement through the corner joint.

(2) By using precast concrete, the contractor was able to reduce (a) rehabilitation time, (b) excavation requirements, (c) costs associated with the forming system, (d) congestion at the restricted project site, and, (e) size of the work force. This use of precast concrete panels resulted in a savings of approximately \$185,000.

g. Blue River Channel Project. The Blue River channel modification project was designed to provide flood protection to the Blue River Basin in the vicinity of Kansas City, MO. In an industrial reach of the river, the project consisted of extensive modification of the channel cross section, construction of a floodwall, paving of side slopes, and construction of a 4.6-m (15-ft)-wide by 1.7-m (5.5-ft)-deep low-flow channel for approximately 1.1 km (3,500 ft) of the river. The original design for the low-flow channel consisted of a pair of sheet-pile walls with a cast-in-place concrete strut between the walls. However, there was some concern about this method of construction because of steel smelting slag and other debris embedded in the existing channel. Driving sheet piles through this material could be very expensive or even impossible. Therefore, an alternate design for a precast concrete U-flume was prepared (Figure 8-46).

(1) The estimated cost of construction for the sheet-pile channel was \$6,035,000 compared with \$8,970,000 to \$11,651,000 for the precast concrete channel depending on how the river water was handled during instruction. A



Figure 8-42. Structural plan and elevation, Chauncy Run Checkdam No. 2

Government estimate for the total project cost (\$31,852,218) was prepared for the sheet-pile channel only, since it was anticipated to have the lowest total cost.

(2) There were seven bidders on the project and all bids were based on the precast concrete alternate. The bids ranged from \$20,835,073 to \$37,656,066 with four

bids below the Government estimate. The low bidder's estimate for the low-flow channel was \$2,000,245, far less than the Government estimate. A comparison of the bids with the Government estimate revealed that selection of the precast concrete alternate was a major factor leading to the bids being considerably lower than the Government



Figure 8-43. Precast concrete checkdams, Chauncy Run

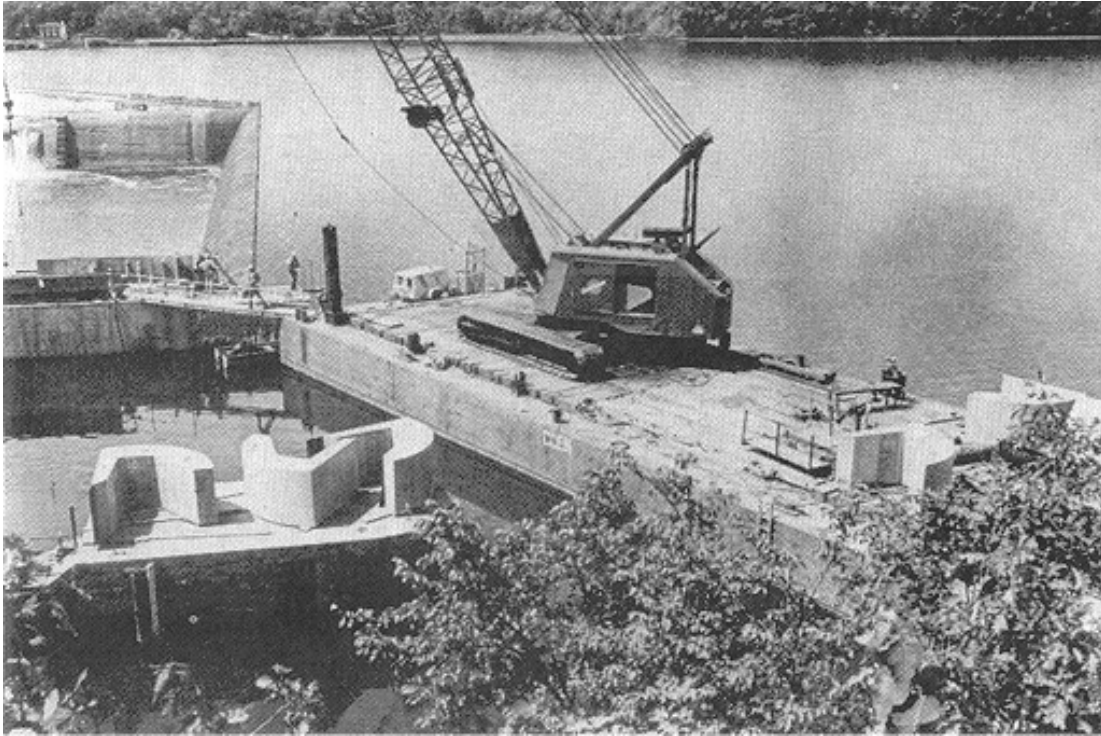


Figure 8-44. Precast pier nose sections ready for underwater installation, Vischer Ferry Dam (Summer 1993)

estimate. For example, the Government estimate contained \$4,753,800 for steel sheetpiling and \$898,365 for the concrete in the strut, while the low bid contained just \$1,057,500 for the precast concrete sections. Also, the Government estimate for water control was \$1,017,000 compared to the low bid of \$400,000. The low bidder's estimate for low-flow costs was \$2 million, far less than the Government estimate.

(3) During discussions some of the bidders indicated that their estimates showed that, compared to the steel sheet-pile structure, it would be approximately \$2 million cheaper to construct the precast concrete U-flume. Their estimates were based on placing 8 to 10 sections per day compared to the Government estimate of 2 sections per day. This difference in production significantly reduced construction time and the cost of water control during construction.

(4) The reinforced-concrete channel sections were precast in 1.5-m (5-ft) lengths which weighed approximately 10,000 kg (11 tons). The concrete in the base slab was placed and cured for 2 days prior to placing the side walls. A concrete compressive strength of 28 MPa (4,000 psi) at 28 days was specified; however, strengths

routinely approached 55 MPa (8,000 psi). Six channel sections were precast daily and stored in the precaster's yard.

(5) The precast sections were shipped, two at a time, by truck to the construction site as required where they were offloaded with a crane and positioned in the channel on a crushed rock subfoundation (Figure 8-47). A total of 700 precast sections were installed with daily placement rates ranging from 12 to 46 sections. The major advantages of precast concrete in this application included low cost, rapid construction, and ease of construction. Also, the use of precast concrete allowed the contractor to divert river water into the channel sections immediately following installation (Figure 8-48). Construction of the low-flow channel was completed in February 1992.

h. Joliet Channel Walls. The walls were constructed in the early 1930's along both banks of the Illinois Waterway through the city of Joliet, IL. The normal pool elevation is 1.2 m (4 ft) below the top of the mass concrete gravity wall on the left bank. The top of the wall is about 2.4 to 9.1 m (8 to 30 ft) higher than the adjacent ground on the landside of the wall.

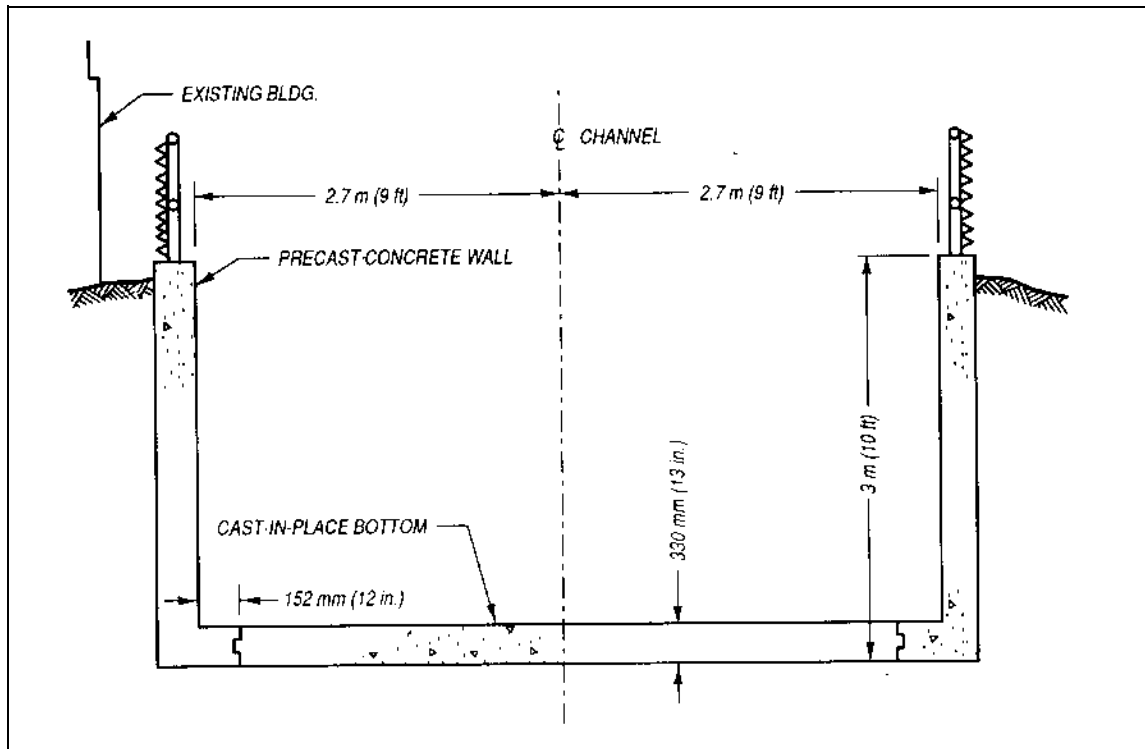


Figure 8-45. Typical channel repair section, Placer Creek (after Hacker 1986)

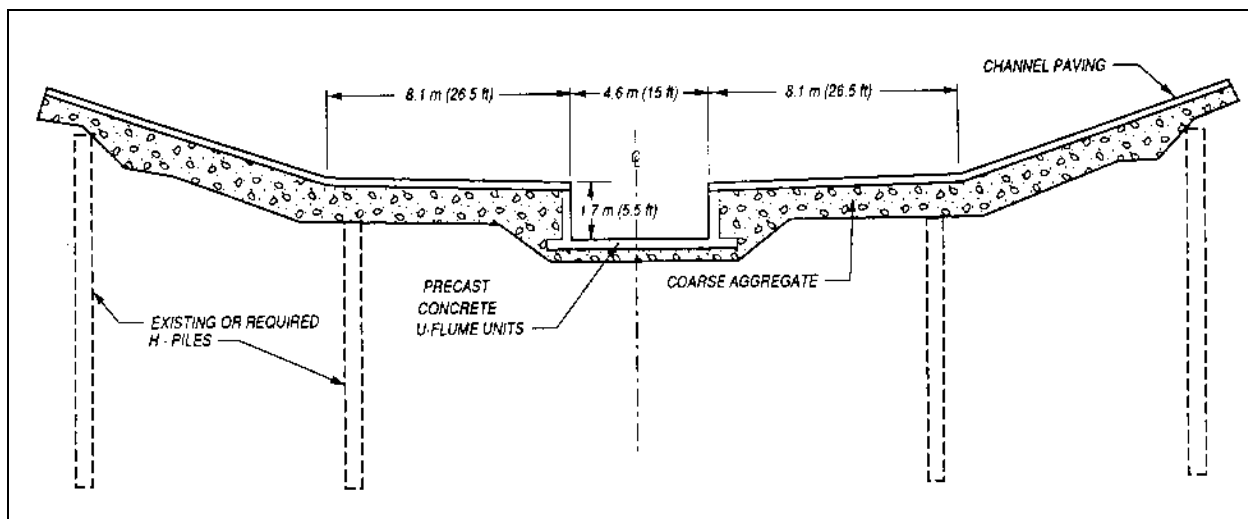


Figure 8-46. Cross section of precast low-flow channel, Blue River Channel (after Mitscher 1992)

(1) A condition survey in 1984 revealed that seepage along monolith and construction joints combined with cycles of freezing and thawing had resulted in extensive deterioration of the exposed concrete to maximum depths of 0.6 m (2 ft). In contrast, those sections of the wall insulated from freezing and thawing by backfill had

escaped deterioration. Stability analyses, which considered the depth of deterioration, confirmed that the gravity walls founded on bedrock remained stable. Therefore, it was decided that any repairs should provide aesthetically acceptable insulation for the exposed concrete walls to

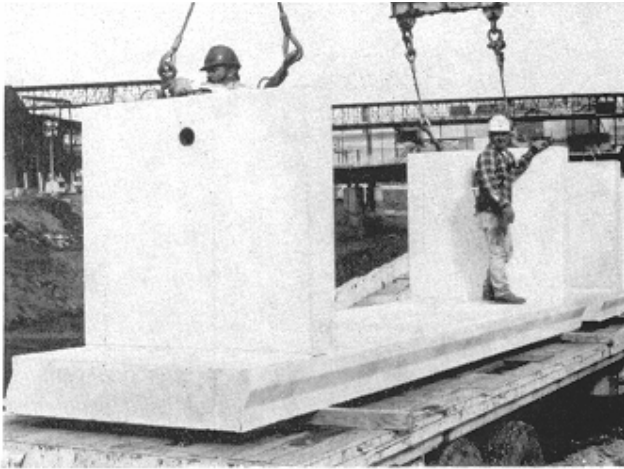


Figure 8-47. Channel section delivery and installation, Blue River Channel



Figure 8-48. Diversion of river water through channel sections, Blue River Channel

reduce the potential for additional freeze-thaw deterioration. Earth backfill, the most economical method of providing insulation, was not feasible in all reaches because of buildings near the wall and other right-of-way restrictions. Consequently, a precast concrete panel system that included insulation and drainage provisions was selected for the repair (Figure 8-49).

(2) The reinforced-concrete panels were precast onsite in horizontal lifts. Form oil was used as a bond breaker to allow separation of panels following curing. As many as six panels were cast on top of each other. Typical panels were 254 mm (10 in.) thick, 5.3 m (17.5 ft) high, and 9.1 m (30 ft) long. Each panel had a groove in one end and a partially embedded waterstop in the other.

(3) The installation sequence began with soil excavation to accommodate the panel footing and a perforated pipe drainage system. A 76-mm (3-in.)-diam pipe was installed vertically in the cast-in-place footing on 4.6-m (15-ft) centers. The pipes were used to collect any water seeping through joints in the wall and convey it into the subsurface drainage system.

(4) A 10-mil-thick polyethylene vapor barrier was placed on the existing wall surface; deteriorated concrete was not removed prior to the repair. A 51-mm (2-in.) thickness of extruded insulation was placed on the vapor barrier. The insulation was then covered with another polyethylene vapor barrier. A crane was then used to lift the panels from the horizontal beds to the wall. The panels were positioned so that the groove accommodated the waterstop from the adjacent panel. Expansion-joint material was placed between the panels, and nonshrink grout was placed in the groove to encapsulate the waterstop. A cast-in-place concrete cap on top of the precast panels minimizes moisture intrusion into the repair.

(5) The total length of the precast concrete repair was 349 m (1,145 ft). The contractor's bid price for the wall repair including drainage system, concrete footing, anchors, insulation, vapor barriers, and concrete cap was \$207 per sq m (\$19 per sq ft). The economical, aesthetically pleasing repair (Figure 8-50) was completed in August 1987 and the repair continues to perform satisfactorily.

i. Ulsterville Bridge. Because this bridge spans a Class 1 trout stream in Ulsterville, NY, it was necessary to minimize onsite construction activities. Consequently,

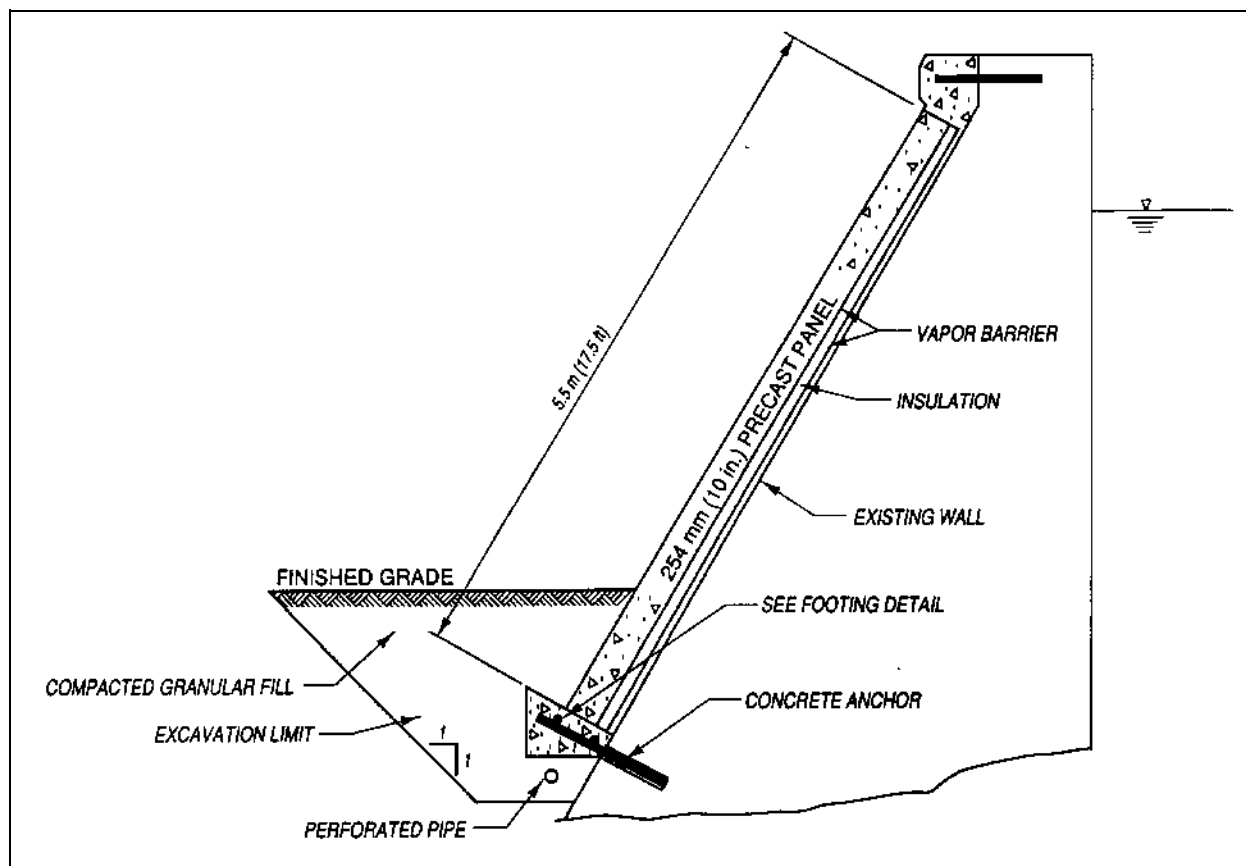


Figure 8-49. Typical repair section, Joliet Channel Wall

precast concrete components were used in September 1989 to construct the bridge abutments and deck.

(1) Precast modules of reinforced concrete stacked on a base slab were used to construct the abutments (Figure 8-51). The modular units were precast in "startup" wood molds, which resulted in some minor fitting problems in the field; however, adjustments were made without the construction being interrupted. Once the modules were properly positioned and aligned, the back cavities in the units were filled with select granular backfill. Vertical reinforcing steel was inserted into the front cavities and these cavities were filled with cast-in-place concrete to form a sealed, monolithic-like front wall (Figure 8-52).

(2) Once the abutments were completed, the precast deck was installed. The composite deck units were precast upside down in forms suspended from wide-flange steel girders. Stud shear connectors were welded to the girders. This technique uses the weight of the forms and the concrete to produce a prestressed effect on the girders.

Another result of the upside down casting is that the densest, least permeable concrete is on the wearing surface. When the cured deck units are turned over, the concrete is precompressed which increases its resistance to cracking. The deck units were placed with a crane; steel diaphragms were installed between the units, and then all longitudinal joints were sealed with grout. The bridge was ready for traffic as soon as construction was complete (Figure 8-53).

(3) Since all components were precast, a small local contractor was able to complete the project within a few days. The cost of the bridge was very competitive with alternate construction procedures and there was minimal environmental impact on the existing trout stream.

j. Summary. The use of precast concrete in repair and replacement of civil works structures has increased significantly in recent years and this trend is expected to continue. A review of these applications shows that, compared with cast-in-place concrete, precasting offers a

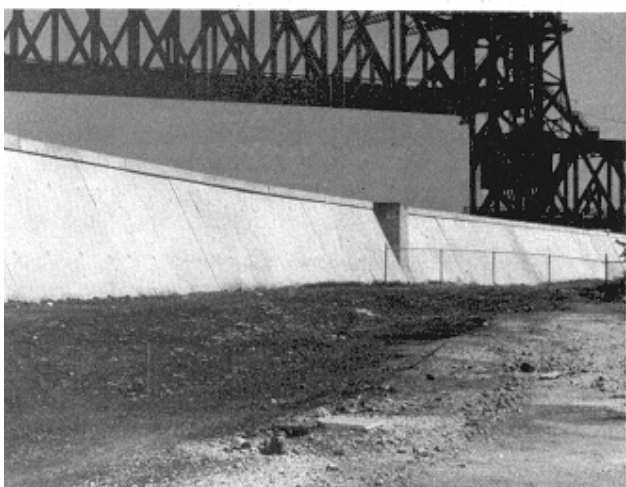
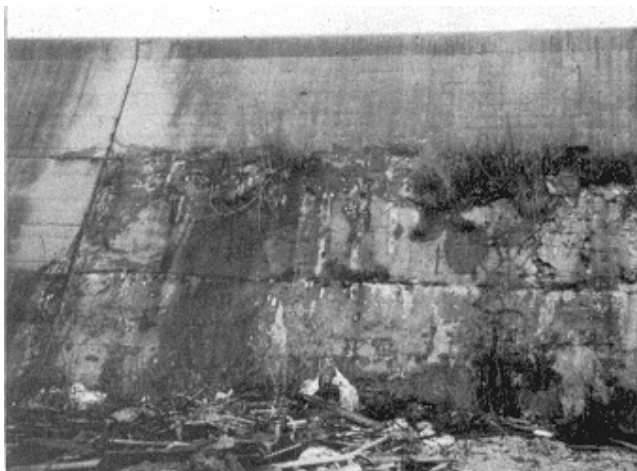


Figure 8-50. Joliet Channel Wall before and after repair

number of advantages including ease of construction, rapid construction, high quality, durability, and economy.

(1) Precasting minimizes the impact of adverse weather. Concrete fabrication in a precaster's plant can continue in winter weather that would make onsite cast-in-place concrete production cost prohibitive or impossible. Also, precast concrete can be installed underwater and in weather conditions where construction with conventional cast-in-place concrete would be impractical.

(2) Concentrating construction operations in the precaster's plant significantly reduces the time and labor required for onsite construction. Reducing onsite construction time is a major advantage in repair of hydraulic structures such as locks and dams where delays and shut-downs can cause significant losses to the users and

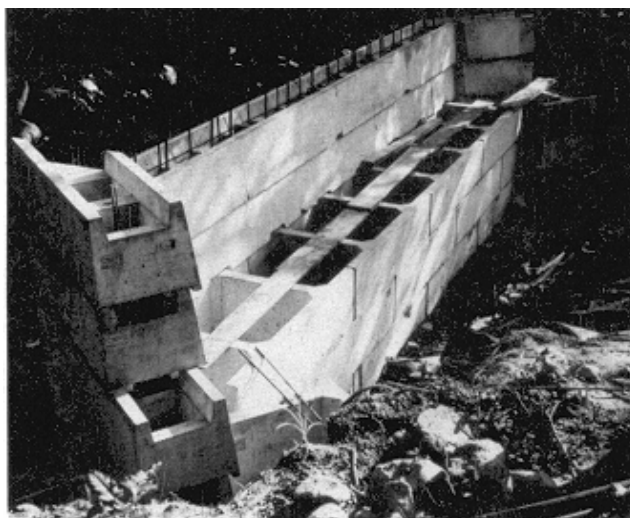


Figure 8-51. Top view of precast abutment prior to backfilling, Ulsterville Bridge

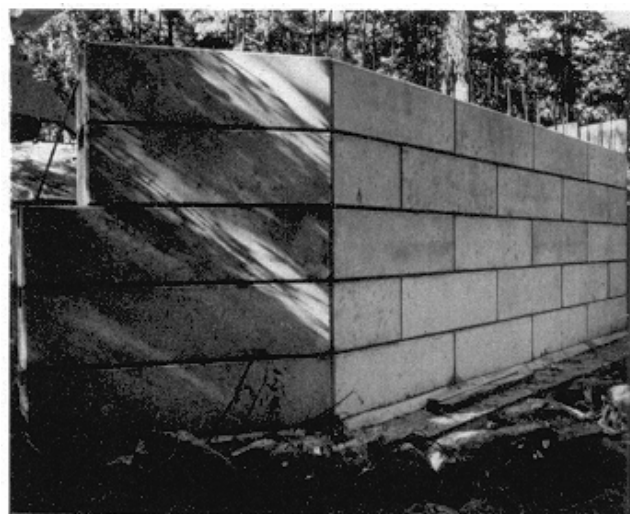


Figure 8-52. Front view of precast abutment prior to placing concrete cap, Ulsterville Bridge

owners. Also, rapid construction minimizes the potential for adverse environmental impact in the vicinity of the project site.

(3) Although the quality and durability of precast concrete is not necessarily better than concrete cast in place at the project site, a qualified precaster usually has the advantages of a concentrated operation in a fixed plant with environmental control; permanent facilities for forming, batching, mixing, placing, and curing; well



Figure 8-53. Precast abutment and deck units, Ulsterville Bridge

established operating procedures, including strict quality control; and, personnel with experience in the routine tasks performed on a daily basis. Also, precasting makes it possible to inspect the finished product prior to its incorporation into the structure.

(4) Ease of construction, rapid construction, and repetitive use of formwork all contribute to lower construction costs with precast concrete. Also, underwater installation of precast concrete eliminates the significant costs associated with dewatering of a hydraulic structure so that conventional repairs can be made under dry conditions. As the number of qualified precast suppliers continues to increase and as contractors become more familiar with the advantages of precast concrete, it is anticipated that the costs of precast concrete will be further reduced.

8-6. Underwater Repairs

Dewatering a hydraulic structure so that repairs can be made under dry conditions is often (a) difficult, and in some cases, practically impossible, (b) disruptive to project operations, and (c) expensive. For example, costs to dewater a stilling basin can exceed \$1 million, and the average cost to dewater is more than 40 percent of the total repair cost (McDonald 1980). Consequently, studies were conducted as part of the REMR research program to develop improved materials and techniques for underwater repair of concrete. These studies included (a) identification of methods and equipment for underwater cleaning and inspection of concrete surfaces, (b) evaluation of materials and procedures for anchor embedment in

hardened concrete under submerged conditions, (c) development of improved materials and techniques for underwater placement of freshly mixed concrete, and (d) development of prefabricated elements for underwater repair. Underwater inspection of concrete surfaces is covered in Section 2-4. Results of the remaining studies are summarized in the following.

a. Surface preparation. All marine growth, sediments, debris, and deteriorated concrete must be removed prior to placement of the repair material. This surface preparation is essential for any significant bond to occur between the repair material and the existing concrete substrate. Equipment and methods specifically designed for underwater excavation and debris removal are available. Also, a wide variety of underwater cleaning tools and methodologies have been designed specifically for cleaning the submerged portions of underwater structures. The advantages and limitations of each are described in detail by Keeney (1987) and summarized in the following.

(1) Excavation. The three primary methods for excavation of accumulated materials, such as mud, sand, clay, and cobbles, include: air lifting, dredging, and jetting. Selection of the best method for excavation depends on several factors: the nature of the material to excavated; the vertical and horizontal distances the material must be moved; the quantity of the material to be excavated; and, the environment (water depth, current, and wave action). General guidance on the suitability of the various excavation methods is given in Figure 8-54.

(a) Air lifts should be used to remove most types of sediment material in water depths of 8 to 23 m (25 to 75 ft).

(b) Jetting and dredging techniques, or combinations thereof, are not limited by water depth.

(2) Debris removal. The primary types of debris that accumulate in hydraulic structures, such as stilling basins, include cobbles, sediment, and reinforcing steel. Cobbles and sediment can be removed with one of the excavation techniques discussed in the previous section. Removal of exposed reinforcing steel often requires underwater cutting of the steel. There are three general categories of underwater steel cutting techniques. The two most common techniques are mechanical and thermal.

(a) Equipment used for mechanical cutting includes portable, hydraulically powered shears and bandsaws.

Excavation Factor	Excavation Method		
	Air Lift	Jet	Dredge
Type of seabed material	mud, sand, silt, clay, cobbles	mud, sand, silt, clay	mud, sand, silt, clay
Water depth	8 to 23 m (25 to 75 ft)	unlimited	unlimited
Horizontal distance material moved	short	short	short to long
Vertical distance material moved	short to long	short	short to medium
Quantity of material excavated	small to large	small to medium	small to medium
Local current	not required	required	not required
Topside equipment required	compressor	pump	pump
Shipped space/weight	large	small	medium

Figure 8-54. Guidance on excavation techniques (Keeney 1987)

(b) Three thermal techniques are recommended for underwater cutting: oxygen-arc cutting, shielded-metal-arc cutting and gas cutting. Oxygen cutting is the preferred technique for Navy Underwater Construction Team diver operations.

(c) The technology exists for underwater abrasive-jet cutting systems, and commercially available equipment is evolving.

(3) Cleaning. There are three general types of cleaning tools: hand tools, powered hand tools, and, self-propelled cleaning vehicles. Hand tools include conventional devices such as scrapers, chisels, and wire brushes. Powered hand tools include rotary brushes, abrasive discs, and water-jet systems. Self-propelled cleaning vehicles are large brush systems that travel along the work surface on wheels. The types of cleaning tools recommended for different types of material, fouling, and surface area are shown in Figure 8-55.

(a) On large and accessible concrete surfaces, a self-propelled vehicle can be used to quickly and effectively remove light to moderate marine and freshwater fouling.

For areas that are not large enough to justify the use of a self-propelled vehicle, hydraulically powered hand tools, such as rotary cutters, can efficiently remove all fouling from concrete surfaces.

(b) A high-pressure waterjet is the best tool to use in obstructed or limited access areas. A high-pressure, high-flow system can be used to remove most types of moderate to heavy fouling. A high-pressure, low-flow system may be required to clean an area that is difficult or impossible to reach with a high-flow system because of the retrojet.

(c) Because of their low cleaning efficiency, hand tools should be used only where there is light fouling or spot cleaning is to be done in limited areas.

b. Anchors. Repairs are often anchored to the existing concrete substrate with dowels. Anchors are particularly necessary in areas where it is difficult to keep the concrete surface clean until the repair is placed. Anchor systems are categorized as either cast-in-place (anchors installed before the concrete is cast) or

Fouling	Size	Material		
		Concrete	Steel	Timber
Light	Massive	Self-propelled vehicles		Waterjets
	Large	Waterjets and hand-held power tools*		
	Limited Access	High-pressure waterjets		
Moderate	Massive	Self-propelled vehicles		Waterjets/ power tools
	Large	Power tools/ waterjets	**	Power tools/ waterjets
	Limited Access	High-pressure waterjets		
Heavy	Massive	Self-propelled vehicles		N/A
	Large	Power tools	Power tools/ waterjets	Power tools
	Limited Access	High-pressure waterjets		

Notes:

- * Hand tools for limited spot cleaning of light and loose fouling.
- ** Abrasive waterjets for paint removal or bare metal finish on steel structures.

Figure 8-55. Guidance on cleaning tools (Keeney 1987)

postinstalled (anchors installed in holes drilled after the concrete has hardened). Since most of the anchors used in concrete repair are postinstalled, anchors can be classified as either grouted or expansion systems.

(1) Expansion anchors are designed to be inserted into predrilled holes and then expanded by either torquing the nut, hammering the anchor, or expanding into an undercut in the concrete. These anchors transfer the tension load from the anchor to the concrete through friction or keying against the side of the drill hole. Detailed descriptions of the various types of expansion anchors are included in ACI 355.1R.

(2) Grouted anchors include headed or headless bolts, threaded rods, and deformed reinforcing bars. They are embedded in predrilled holes with either cementitious or

polymer materials. Cementitious materials include portland-cement grouts, with or without sand, and other commercially available premixed grouts. Polymer materials are generally two-component compounds of polyesters, vinylesters, or epoxies. These resins are available in four forms: tubes or "sausages," glass capsules, plastic cartridges, or bulk. Setting times for polymer materials are temperature dependent and can vary from less than a minute to several hours depending on the formulation.

(3) The effectiveness of neat portland-cement grout, epoxy resin, and prepackaged polyester resin in embedding anchors in hardened concrete was evaluated under a variety of wet and dry installation and curing conditions (Best and McDonald 1990b). Pullout tests were conducted at eight different ages ranging from 1 day to

32 months. Creep and durability tests were also conducted.

(a) Beyond 1 day, all pullout strengths were approximately equal to the ultimate strength of the reinforcing-bar anchor when the anchors were installed under dry conditions, regardless of the type of embedment material or curing conditions. With the exception of the anchors embedded in polyester resin under submerged conditions, pullout strengths were essentially equal to the ultimate strength of the anchor when the anchors were installed under wet or submerged conditions. The overall average pullout strength of anchors embedded in polyester resin under submerged conditions was 35 percent less than the strength of similar anchors installed and cured under dry conditions. The largest reductions in pullout strength, approximately 50 percent, occurred at ages of 6 and 16 months. Although the epoxy resin performed well in these tests when placed in wet holes, it should be noted that the manufacturer does not recommend placement under submerged conditions.

(b) Creep tests were conducted by subjecting pullout specimens to a sustained load of 60 percent of the anchor-yield strength and periodically measuring anchor slippage at the end of the specimen opposite the loaded end. After 6 months under load, anchors embedded in portland-cement grout and epoxy resin that were installed and tested under wet conditions exhibited low anchor slippage, averaging 0.071 and 0.084 mm (0.0028 and 0.0033 in.), respectively, or two to four times higher than results under dry conditions. Anchors embedded in polyester resin, installed and cured under submerged conditions, exhibited significant slippage; in fact, in one case the anchor pulled completely out of the concrete after 14 days under load. After 6 months under load, the two remaining specimens exhibited an average anchor slippage of 2.09 mm (0.0822 in.), approximately 30 times higher than anchors embedded in portland-cement grout under the same conditions.

(c) Long-term durability of the embedment materials was evaluated by periodic compressive strength tests on 51-mm (2-in.) cubes stored both submerged and in laboratory air. After 32 months, the average compressive strength of polyester-resin and epoxy-resin specimens stored in water was 37 and 26 percent less, respectively, than that of companion specimens stored in air. The strength of portland-cement grout cubes stored in water averaged 5 percent higher than that of companion specimens stored in air during the same period.

(4) The performance of anchors embedded in vinyl-ester resin, prepackaged in glass capsules, was also evaluated under dry and submerged conditions (McDonald 1989). Pullout tests were conducted at four different ages ranging from 1 to 28 days. The tensile capacity of anchors embedded under submerged conditions was approximately one-third that of similar anchors embedded in dry holes.

(5) The reduced tensile capacity of anchors embedded in concrete under submerged conditions with prepackaged polyester-resin and vinylester-resin cartridges is primarily attributed to the anchor installation procedure. Resin extruded from dry holes during anchor installation was very cohesive, and a significant effort was required to obtain the full embedment depth. In comparison, anchor installation required significantly less effort under submerged conditions. Also, the extruded resin was much more fluid under wet conditions, and the creamy color contrasted with the black resin extruded under dry conditions. Although insertion of the adhesive capsule or cartridge into the drill hole displaces the majority of the water in the hole, water will remain between the walls of the adhesive container and the drill hole. Insertion of the anchor traps this water in the drill hole and causes it to become mixed with the adhesive, resulting in an anchor with reduced tensile capacity. An anchor-installation procedure that eliminates the problem of resin and water mixing in the drill hole is described by McDonald (1990).

(a) In the revised installation procedure (Figure 8-56), a small volume of adhesive was injected into the bottom of the drill hole in bulk form prior to insertion of the adhesive capsule. This injection was easily accomplished with recently developed paired plastic cartridges (Figure 8-57) which contained the vinylester resin and a hardener. The cartridges were inserted into a tool similar to a caulking gun which automatically dispensed the proper material proportions through a static mixing tube directly into the drill hole. Once the injection was completed, insertion of a prepackaged vinylester-resin capsule displaced the remainder of the water in the drill hole prior to anchor insertion and spinning.

(b) Anchors installed with the revised procedure exhibited essentially the same tensile capacity under dry and submerged conditions. At 3-mm (0.1-in.) displacement, the tensile capacity of vertical anchors installed with the revised procedure under submerged conditions averaged more than three times greater than that of similar anchors installed with the original procedure. The

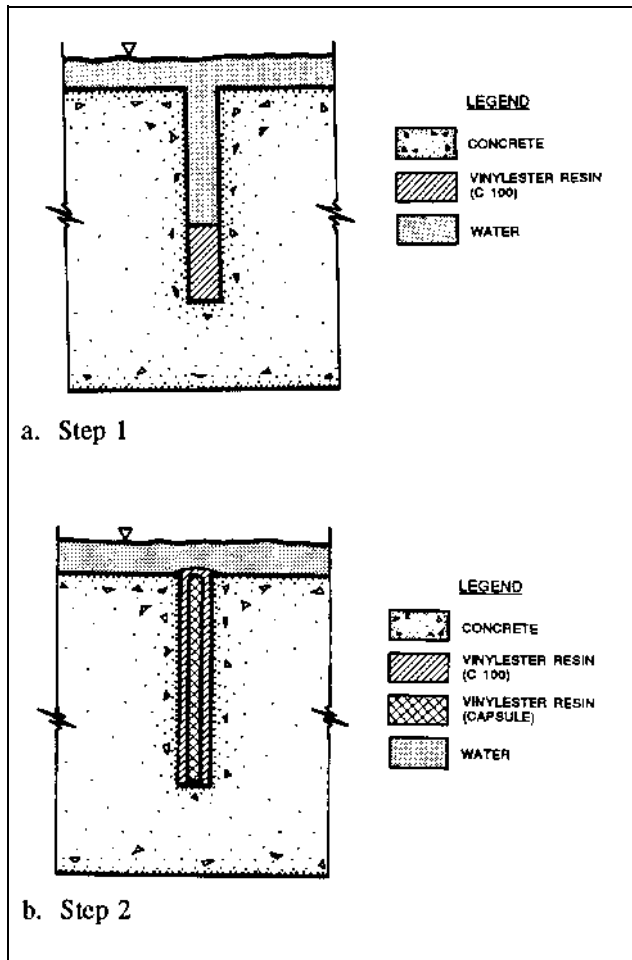


Figure 8-56. Two-step procedure for anchor installation under submerged conditions

ultimate tensile capacity of anchors installed under submerged conditions was near the yield load of the anchors. Also, the difference in tensile capacity between horizontal anchors installed under dry and submerged conditions was less than 2 percent.

(6) Epoxy resins were not prepackaged in “sausage” type cartridges because insertion and spinning of the anchor did not provide adequate mixing. However, with development of the coaxial or paired disposable cartridges with static mixing tubes, a number of suppliers are presently marketing epoxies for anchor embedment under submerged conditions. Also, some suppliers contend that insertion of the prepackaged capsule in the second step of the two-step installation procedure can be eliminated by injecting additional epoxy. However, preliminary results of current tests indicate that anchors installed by epoxy injection alone perform very poorly.

(7) A “nonshrink cementitious anchor cartridge” was recently introduced on the market. According to the supplier, the cartridge contains a fast-setting cementitious compound encased in a unique envelope, which when immersed in water will allow controlled wetting of the contents, forming a thixotropic grout. Results of preliminary tests on anchors installed with this system under submerged conditions are very favorable.

(8) Pending completion of current tests, the two-step anchor installation procedure (Figure 8-56) should be followed when prepackaged vinyl ester resin is to be used as an embedment material for short (less than 381-mm (15-in.) embedment length) steel anchors in hardened

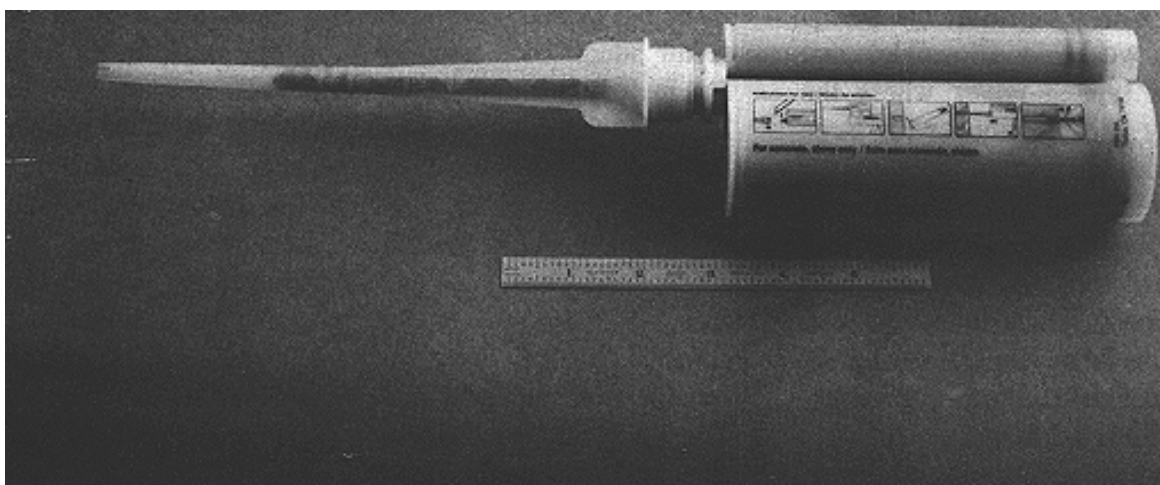


Figure 8-57. Paired disposable cartridges and static mixing tube

concrete under submerged conditions. Similar anchors embedded in neat portland-cement grout exhibit excellent performance when the grout is allowed to cure for a minimum of 3 days prior to loading. The ability of the anchor system including any embedment material, to perform satisfactorily under the exposure conditions, particularly creep and fatigue, should be evaluated during design of the repair.

c. *Materials.* Cast-in-place concrete and prefabricated elements of concrete and steel have been used successfully in underwater repair of hydraulic structures. Each material has inherent advantages and limitations which should be considered in design of a repair for specific project conditions.

(1) Cast-in-place concrete. Successful underwater concrete placement requires that the fresh concrete be protected from the water until it is in place and begins to stiffen so that the cement and other fines cannot wash away from the aggregates. This protection can be achieved through proper use of placing equipment, such as tremies and pumps (ACI 304R, and Gerwick 1988). Also, the quality of the cast-in-place concrete can be enhanced by the addition of an antiwashout admixture (AWA) which increases the cohesiveness of the concrete (Khayat 1991, Neeley 1988, and Neeley, Saucier, and Thornton 1990). The purpose, types, and functions of AWA's for concrete used in underwater repairs is described in REMR Technical Note CS-MR-7.2 (USAEWES 1985f).

(a) Concrete mixtures for underwater placement must be highly workable and cohesive. The degree of workability and cohesiveness can vary somewhat depending upon the type of placing equipment being used and the physical dimensions of the placement area. For example, massive and confined placements, such as cofferdams, or bridge piers, can be completed with a conventional tremie concrete mixture that is less workable (flowable) and cohesive than a mixture for a typical repair where the concrete is placed in relatively thin sections with large surface areas. An AWA should be used to enhance the cohesiveness of concrete that must flow laterally in thin lifts for a substantial distance. Silica fume should be used to enhance the hardened properties of concrete subjected to abrasion-erosion. The addition of silica fume will also increase the cohesiveness of the fresh concrete mixture. Guidance on proportioning concrete mixtures for underwater placement is given in EM 1110-2-2000.

(b) The tremie method has been successfully used for many years to place concrete underwater (ACI 304R).

The tremie pipe must be long enough to reach from above water to the location underwater where the concrete is to be deposited. Concrete flows through the tremie, the lower end of which is embedded in a mound of the fresh concrete so that all subsequent concrete flows into the mound and is not exposed directly to the surrounding water. Tremie concrete mixtures must be fully protected from exposure to water until in place. AWA's can be used in tremie concrete but are not necessary, although their use will enhance the cohesiveness and flowability of the concrete. If an AWA is used, embedment of the tremie in the fresh concrete is still desirable, but some exposure of the fresh concrete to water may be permitted. Free-fall exposure of the concrete through water is not recommended under any conditions. Usually the tremie is maintained in a vertical position. However, recent research has indicated that inclining the tremie to approximately 45 deg is effective when placement conditions require the concrete to flow laterally several feet in thin layers (Khayat 1991).

(c) The Hydrovalve method and the Kajima Double Tube tremie method (Gerwick 1988) are each variations of the traditional tremie. Each uses a flexible hose that collapses under hydrostatic pressure and thus carries a controlled amount of concrete down the hose in slugs. This slow and contained movement of the concrete helps to prevent segregation and is particularly useful in placing conventional tremie concrete. These methods are reliable, inexpensive, and can be used by any contractor with personnel experienced in working underwater.

(d) In recent years, pumping has become preferable to the tremie method for placing concrete underwater. There are fewer transfer points for the concrete, the problems associated with gravity feed are eliminated, and the use of a boom permits better control during placement. These advantages are especially important when concrete is being placed in thin layers, as is the case in many repair situations. A concrete pump was effectively used for underwater repair of erosion damage at Red Rock Dam (Neeley and Wickersham 1989). A diver controlled the end of the pump line, keeping it embedded in the mass of newly discharged concrete and moving it around to completely fill the repair area. The effects of the AWA used were apparent; even though the concrete had a slump of 229 mm (9 in.), it was very cohesive. The concrete pumped very well and, according to the diver, self-levelled within a few minutes following placement. The diver also reported that the concrete remained cohesive and exhibited very little loss of fines on the few occasions when the end of the pump line kicked out of the concrete. The total cost of the repair was \$128,000.

In comparison, estimated costs to dewater alone for a conventional repair ranged from \$1/2 to \$3/4 million.

(e) Pneumatic valves attached to the end of a concrete pump line permit better control of concrete flow through the lines and even allow termination of the flow to protect the concrete within the lines and to prevent excessive fouling of the water while the boom is being moved. Some units incorporate a level detector to monitor the concrete placement. This method is considered to be one of the most effective for placing concrete underwater (Gerwick 1988). Guidance on pumped concrete is given in EM 1110-2-2000. Also, ACI 304.2R is an excellent reference.

(f) Preplaced-aggregate concrete is an effective method for repairing large void areas underwater. The coarse aggregate is enclosed in forms, and grout is then injected from the bottom of the preplaced aggregate. To prevent loss of fines and cement at the top of the repair, venting forms have a permeable fabric next to the concrete, backed with a wire mesh, and supported by a stronger backing of perforated steel and plywood. The pressure generated by the grout beneath the forms necessitates doweling to hold down the forms. The grout must have a high fluidity to ensure complete filling of voids. Use of an AWA will lessen the need for the protective top form. Guidance on preplaced-aggregate concrete is given in EM 1110-2-2000 and ACI 304.1R.

(g) Some techniques have been developed, such as buckets, skips, and tilting pallet barges, to place stiff, highly cohesive concrete by dropping the concrete through several feet of water, thus requiring the use of a large amount of AWA. However, until these methods have been fully developed and proved effective, there is a substantial risk that the end result will be a poor quality concrete placement. Therefore, placing concrete by allowing it to free-fall through several feet of water is not recommended.

(2) Prefabricated elements. Precast concrete panels and modular sections have been successfully used in underwater repair of lock walls (Section 8-1), stilling basins, and dams (Section 8-5). Prefabricated steel panels have been used underwater as temporary and permanent top forms for preplaced-aggregate concrete. Also, prefabricated steel modules have been used in underwater repair of erosion damage. Rail and Haynes (1991) concluded that it is feasible to use prefabricated steel, precast concrete, or composite steel-concrete panels in underwater repair of stilling basins. Each material has inherent

advantages, and the following factors should be considered in designing panels for a specific project.

(a) Abrasion resistance. It is the opinion of Rail and Haynes (1991) that the abrasion resistance of steel is far superior to that of concrete; however, no data were given to substantiate this claim. Recent tests by Simons (1992) indicate that the widths and depths of abrasion of concrete mixtures with an average compressive strength of 75 MPa (10,915 psi) at 28 days were 1.6 and 2.3 times higher, respectively, compared to abrasion-resistant steel. However, when the depths of abrasion were compared as a percentage of the thickness for typical panels of each material, it was concluded that the durability of the high-strength concrete was equivalent to that of the abrasion-resistant steel. Guidance on proportioning of high-strength concrete mixtures is given in EM 1110-2-2000. A thorough discussion of high-strength concrete is given in ACI 363R.

(b) Uplift. Uplift forces caused by high-velocity water flowing over the panel surface is a concern in repair of hydraulic structures, particularly stilling basins. The Old River Low Sill Control Structure demonstrated the problem of uplift of steel panels (McDonald 1980). Thirty modules, 7.3 m (24 ft) long and ranging in width from 0.9 to 6.7 m (3 to 22 ft), were prefabricated from 13-mm (1/2-in.)-thick steel plate for the stilling basin repair. After the modules were installed and anchored underwater, the voids between the steel plate and the existing concrete slab were filled with grout. Additional anchors were installed in holes drilled through the grouted modules and into the basin slab to a depth of 0.9 m (3 ft). An underwater inspection of the basin 8 months after the repairs showed that 7 of the 30 modules had lost portions of their steel plate, ranging from 20 to 100 percent of the surface area. A number of anchor bolts were found broken flush with the surface plate or the grout or pulled completely out of the substrate. A second inspection, approximately 2 years after repair, revealed that additional steel plate had been ripped from four of the modules previously damaged, and an additional nine modules had sustained damage. There were only a few remnants of the steel plates when the basin was dewatered 11 years after the repairs. Precast concrete panels, with a minimum thickness of about 100 mm (4 in.), should be less susceptible to these problems. The increased panel stiffness, damping, and bond to the infill concrete will reduce uplift problems.

(c) Anchors. The design of the anchor system should ensure that the prefabricated panels are adequately

anchored to the existing concrete to resist the uplift forces and vibrations created by flowing water. Welding of anchor systems as nearly flush with the prefabricated steel plate surface as possible appears more desirable than raised bolted connections. Preformed, recessed holes for anchors were easily incorporated during precasting of the concrete panels used in underwater repairs at Gavins Point Dam (Section 8-5). The ability of the anchor system, including any embedment material, to perform satisfactorily under the exposure conditions, particularly creep and fatigue, should be evaluated during design of the repair.

(d) Joints. Joint details are important design considerations because once a panel fails and is displaced, adjacent panels are more susceptible to failure as a result of their exposed edges. Consequently, each panel should be designed as an individual repair unit and should not rely on adjacent panels for protection. The vulnerability of joints can be reduced by providing stiffened and recessed panel edges. The joint between the existing concrete and the leading edge of the upstream panels should be designed to provide a smooth transition to the repair section. A general design philosophy should be to minimize the number of joints by using the largest panels practical.

(e) Weight. Panels are usually installed with a barge-mounted crane. Therefore, panel weight can be important, depending on the lifting capacity of available cranes. For panels of a given area, the weight of concrete panels in air will be about four times the weight of steel panels.

(f) Panel supports. Panel supports are bottom-installed platforms or seats which may be required to ensure that panels are placed at the desired elevation and are properly aligned. Rail and Haynes (1991) present several concepts for panel supports, some of which include a means to attach and anchor the panel to resist uplift forces. The potential of these concepts should be evaluated based on specific project requirements.

(g) Infill placement. The physical dimensions of the placement area dictate whether portland-cement grout or concrete is used as the infill material. Grout should be used in those applications that require the infill to flow substantial distances in very thin lifts, whereas pumped concrete or preplaced-aggregate concrete should be used to fill larger voids.

d. Inspection. Inspection of the work as it progresses is important, and there are a number of techniques and procedures for underwater inspection described in Section 2-4. The selection of inspection techniques will

be affected by type, size, location, and environmental conditions of a particular job, along with technical capabilities and limitations and monetary constraints. Visual inspection is usually the first technique considered for underwater inspection. Low-light video is an alternative to the diver-inspector. A diver with two-way communication can position the video camera as directed by the inspector who views the video from the surface. A remotely operated vehicle (ROV) with a video camera can also be used where water currents and turbulence permit use of an ROV. More sophisticated techniques such as echo sounders, side-scan sonar, radar, laser mapping, and high-resolution acoustic mapping are also available. The high-resolution acoustic mapping system appears to be ideal for inspection of underwater repairs.

e. Support personnel/equipment. A qualified diving team is required for underwater repairs. Personnel should be skilled in the operation of construction and inspection equipment to be used during the repair. All personnel should have a thorough understanding of proper working procedures to minimize the risk of an accident. Personnel safety should be given a high priority.

8-7. Geomembrane Applications

Many of the Corps' structures exhibit significant concrete cracking which allows water intrusion into or through the structure. These cracks are the result of a variety of phenomena, including restrained concrete shrinkage, thermal gradients, cycles of freezing and thawing, alkali-aggregate reaction, and differential settlement of the foundation. Water leakage through hydraulic structures can also result from poor concrete consolidation during construction, improperly prepared lift or construction joints, and waterstop failures. When leakage rates become unacceptable, repairs are made. Conventional repair methods generally consist of localized sealing of cracks and defective joints by cementitious and chemical grouting, epoxy injection, or surface treatments. Even though localized sealing of leaking cracks and defective joints with conventional methods has been successful in some applications, in many cases some type of overall repair is still required after a few years. Consequently, the potential for geomembranes in such repairs was evaluated as part of the REMR Research Program.

a. Background. Various configurations of geomembranes have been used as impervious synthetic barriers in dams for more than 30 years. Generally, membranes are placed within an embankment or rockfill dam as part of the impervious core or at the upstream face of embankment, rockfill, and concrete gravity dams.

In recent years, geomembranes have been increasingly used for seepage control in a variety of civil engineering structures, including canals, reservoirs, storage basins, dams, and tunnels. Geomembranes have also been used successfully to resurface the upstream face of a number of old concrete and masonry dams, particularly in Europe.

b. Definition. Consistent with the International Commission on Large Dams (ICOLD 1991) the term "geomembrane" is used herein for polymeric membranes which constitute a flexible, watertight material with a thickness of one-half to a few millimeters. A wide range of polymers, including plastics, elastomers, and blends of polymers, are used to manufacture geomembranes.

c. Fabrication. Since the existing concrete or masonry surfaces to receive the geomembrane are usually rough, a geotextile is often used in conjunction with the geomembrane to provide protection against puncturing. For example, the geocomposite SIBELON CNT consists of a nonwoven, needle-punched polyester geotextile which is bound to one side of the flexible PVC geomembrane by heating during the extrusion process. The geotextile is also designed to function as a drain to evacuate any water between the concrete and the geomembrane. The PVC and geotextile layers are 2.5 and 1.5 mm (0.1 and 0.6 in.) thick, respectively, and weigh 3,250 and 500 g/sq m (6 and 0.9 lb/sq yd), respectively. The geocomposite is manufactured in rolls with a minimum width of 2.05 m (6.7 ft) and lengths dependent upon the specific application. In dam applications, each roll is long enough to

cover the height of the upstream face where it is to be installed, thus avoiding horizontal welds.

d. Installation. In early applications, geomembranes were attached directly to the upstream face of concrete dams with nails or adhesives. In more recent applications, geocomposites have been installed with stainless steel profiles anchored to the face of the dam. This system (SIBELON SYSTEMS DAMS/CSE) consists of two vertical U-shaped sections fabricated to fit one inside the other to form a continuous rib (Figure 8-58). In addition to a uniform, continuous anchorage of the geocomposite, the system also allows the geomembrane to be pretensioned, thus eliminating the problem of sagging caused by the weight of the geomembrane.

(1) Profiles are not installed near vertical monolith joints to allow the elastic geocomposite to accommodate longitudinal joint movements. The vertical anchorage and tensioning profiles are fabricated in 1.4-m (4.6-ft) lengths with slots for the threaded rods to allow relative vertical displacements between adjacent monoliths.

(2) Any water which might collect behind the geocomposite is conveyed, at atmospheric pressure, along the profiles to the heel of the dam where it can be collected in drainage pipes. A high-density polyethylene geonet, 4 mm (0.016 in.) thick with a diamond-shaped mesh, can be installed behind the geocomposite to increase the drainage capacity of the system. The perimeter of the membrane system is sealed against the concrete face with

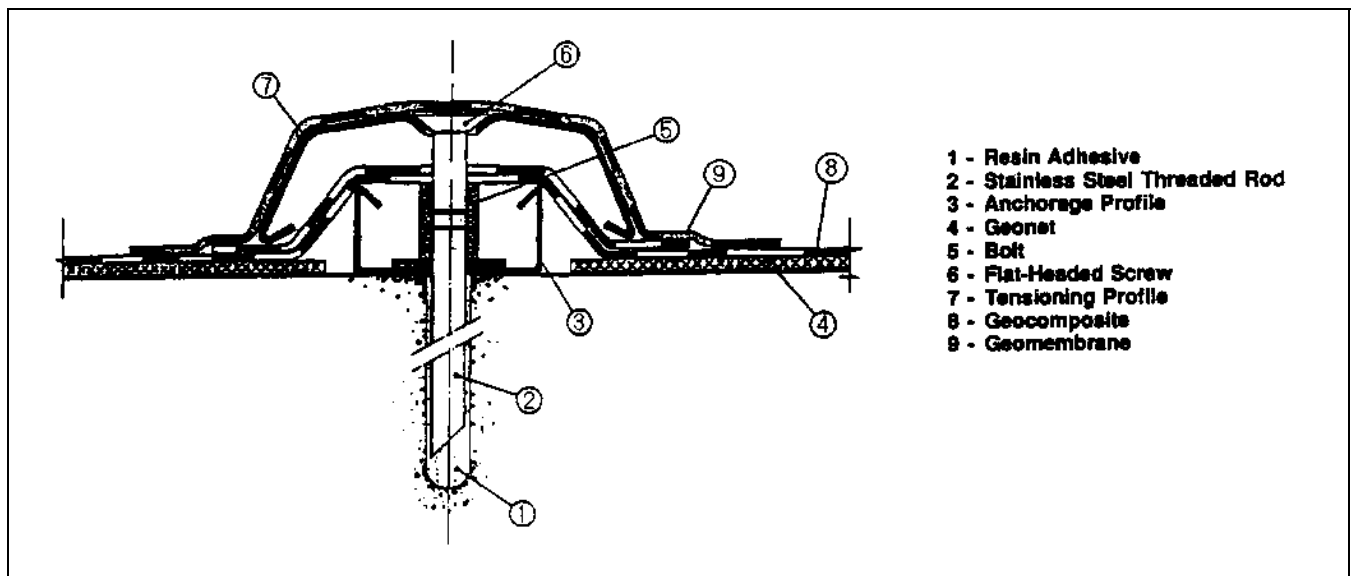


Figure 8-58. Detail of vertical anchorage and drainage profile

stainless steel profiles to prevent intrusion of reservoir water. A flat profile is used for the horizontal seal at the crest of the dam and C-shaped profiles (Figure 8-59) are used along the heel of the dam and the abutments.

e. Applications. Geomembranes have been used to rehabilitate concrete, masonry and rockfill, gravity dams and concrete arch dams including multiple and double curvature arches. Geomembranes have also been used to rehabilitate reservoirs and canals and to provide a water-retention barrier on the upstream face of new dams constructed with roller compacted concrete.

(1) Lake Baitone Dam. The first Italian application of a geomembrane for rehabilitation purposes was at the 37-m (121-ft)-high Lake Baitone Dam (Cazzuffi 1987). The upstream face of the stone masonry and cement mortar structure, completed in 1930, was lined with a series of vertical, semicircular concrete arches with an internal diameter of 1.7 m (5.6 ft). Deterioration of the concrete surfaces required rehabilitation in 1970 with a 2-mm (.08-in.)-thick polyisobutylene geomembrane applied directly on the arches with an adhesive, without any external protection. After more than 20 years in service, the geomembrane exhibited good adhesion, and its

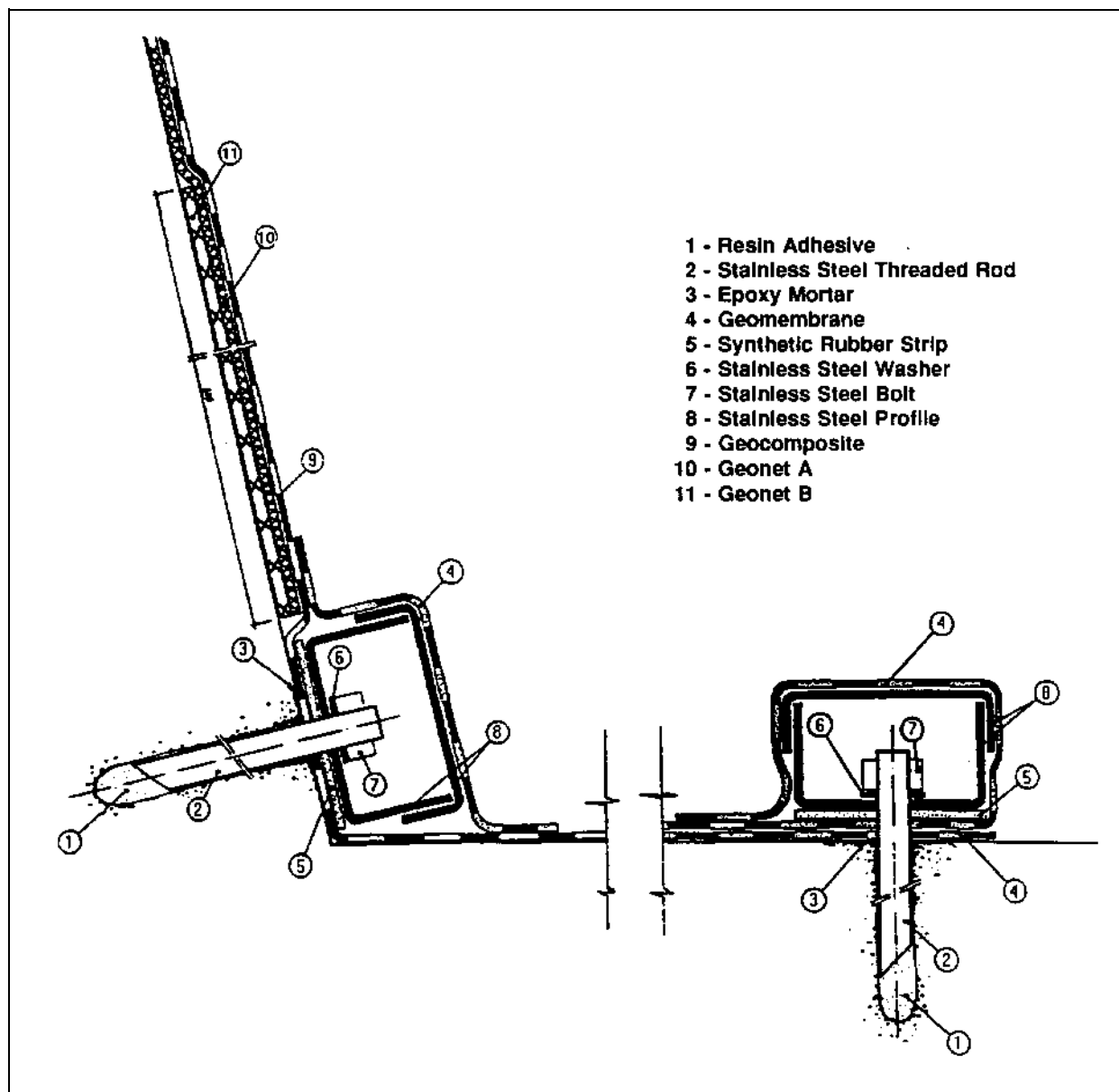


Figure 8-59. Detail of sealing profiles

performance was considered to be satisfactory, although there had been some damage caused by heavy ice formation which was quickly and easily repaired. However, when similar repairs were later made on concrete gravity dams, the results were clearly negative (Monari and Scuero 1991). This difference in performance was attributed to the extensive network of cracks in the thin arches which provided for natural drainage of vapor pressure in contrast to the limited drainage associated with the thicker gravity sections with minimal cracking. This experience and subsequent laboratory tests led to the conclusion that geomembrane repair systems should be mechanically anchored and must permit drainage of any water which might be present behind the geomembrane.

(2) Lake Nero Dam. This 40-m (131-ft)-high concrete gravity dam with a crest length of 146 m (479 ft) was completed in 1929 near Bergamo, Italy. Over the years, various repairs, including grouting of the bedrock and shotcreting of the upstream face were conducted. However, these attempts to eliminate leakage through the dam and foundation and deterioration of the concrete caused by aggressive water and cycles of freezing and thawing proved to be temporary or inadequate.

(a) In 1980, a geocomposite was installed on the upstream face of the dam and anchored with steel profiles (Monari 1984). Self-hoisting platforms secured to the dam's crest were used in the installation (Figure 8-60) which required 90 days to complete. The geocomposite was unrolled from the top of the dam down to the base. Adjacent sheets were overlapped and the vertical joints were welded prior to horizontal prestressing. The lower ends of the sheets were anchored to the base of the dam with metal plates.

(b) Installation of the geocomposite reduced leakage from 50 l/sec to 0.27 l/sec with only 14 percent of the leakage through the geocomposite. After 10 years in service, the geocomposite had required no maintenance nor had the lining lost any of its original efficiency (Monari and Scuero 1991).

(3) Cignana Dam. This 58-m (190 ft)-high concrete gravity dam with a crest length of 402 m (1,319 ft) was completed in 1928. Located at an elevation of 2,173 m (7,129 ft) above sea level, the concrete exhibited evidence of considerable freeze-thaw degradation. Also, the dam had major leakage problems despite extensive maintenance efforts, including application of paint-on resin membranes. A 2.5-mm (0.1-in.)-thick geocomposite was

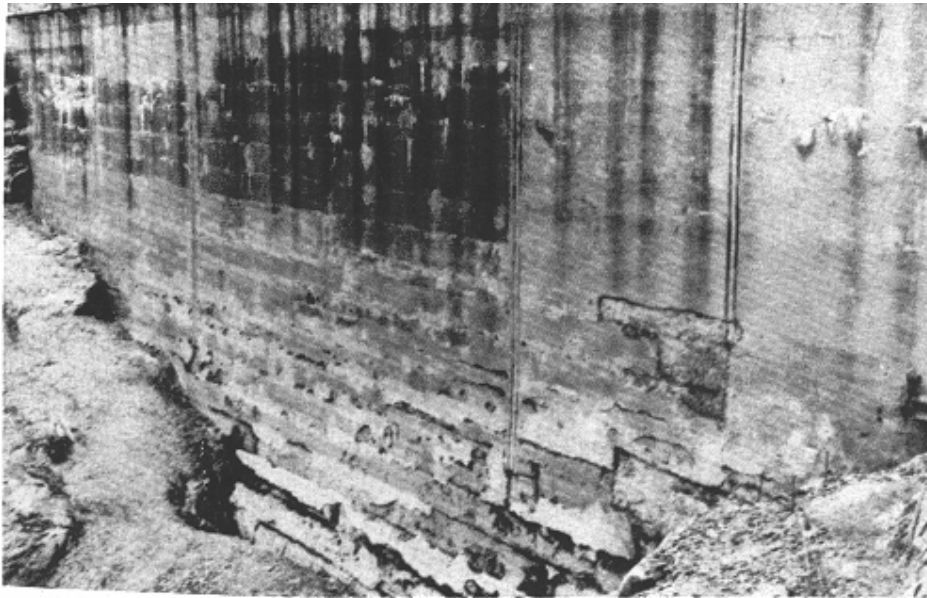
used to rehabilitate the dam in 1987. The repair was similar to that of Lake Nero Dam except that the vertical steel profiles were embedded in a layer of reinforced shotcrete used to resurface the upstream face of the dam. After removal of deteriorated concrete, a layer of shotcrete was applied, vertical profiles were installed, and the surface between adjacent profiles was made level by filling with a second layer of shotcrete (Figure 8-61).

(4) Pracana Dam. This 65-m (213-ft)-high concrete buttress dam with a crest length of 240 m (787 ft) is located on the Ocreza River in Portugal. Cracking occurred soon after construction, causing leakage at the downstream face. An attempt to reduce leakage by grouting the cracks did not result in a durable solution to the problem; therefore, a comprehensive rehabilitation was initiated in 1992. Application of a geocomposite to the upstream face of the dam was a major component of the rehabilitation.

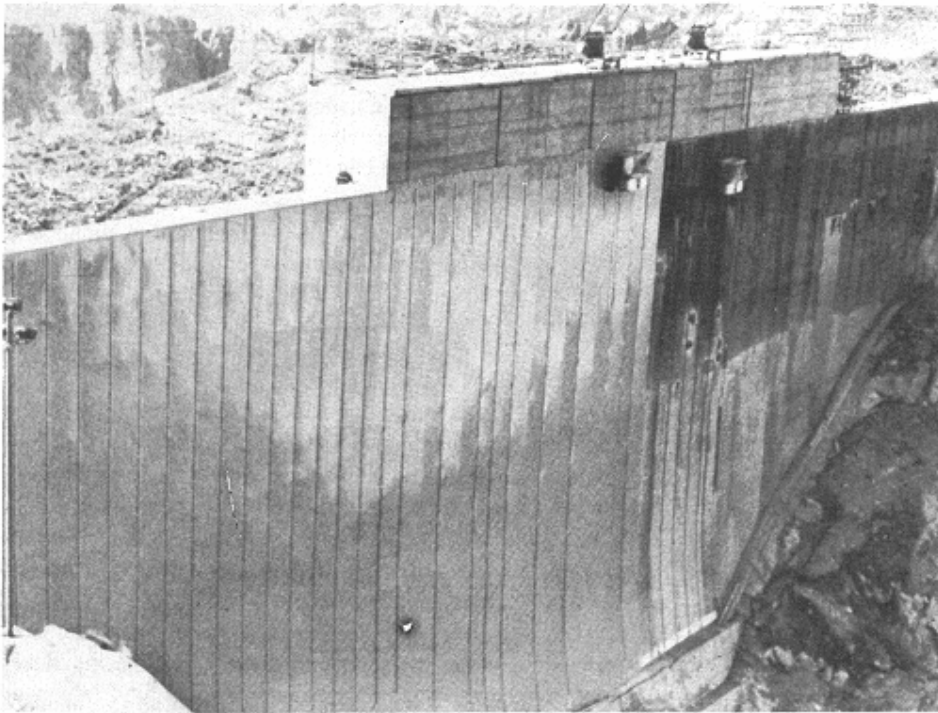
(a) A high-density polyethylene geonet was attached to the upstream face to increase the capacity of the geomembrane system to drain moisture from the concrete and any leakage through the geocomposite. An additional geonet was installed for a height of about 1 m (3 ft) above the heel of the dam to assist in conveying the water to drainage pipes. Ten near-horizontal holes were drilled from the heel of the dam to the downstream face to accommodate the drainage pipes. The geomembrane system was divided into six independent compartments from which water is piped to the downstream face for volumetric measurements. This design provides improved monitoring of the drainage behind the geomembrane.

(b) Approximately 8,000 sq m (9,600 sq yd) of geocomposite were installed with steel profiles shown in Figures 8-58 and 8-59. Vertical joints between rolls of the geocomposite and geomembrane strips covering the profiles (Figure 8-62) were welded to provide a continuous water barrier on the upstream face of the dam. A hot-air jet was used to melt the plastic, and the two surfaces were then welded together by hand pressure on a small roller.

(c) A wire system was installed behind the geocomposite to aid in future monitoring and maintenance of the geomembrane. A sensor sliding on the surface of the geomembrane can detect any anomalies in the electrical field thus locating any discontinuities in the geomembrane.



a. Prior to rehabilitation



b. During rehabilitation

Figure 8-60. Upstream face of Lake Nero Dam

f. Summary.

(1) Geomembranes and geocomposites have been installed on the upstream face of more than 20 old concrete dams during the past 23 years. The success of these systems in controlling leakage and arresting concrete deterioration and the demonstrated durability of these materials are such that these systems are considered competitive with other repair alternatives.

(2) With a few exceptions, geomembrane installations to date have been accomplished in a dry environment by dewatering the structure on which the geomembrane is to be installed. Dewatering, however, can be extremely expensive and in many cases may not be possible because of project constraints.

(3) A geomembrane system that could be installed underwater would have significantly increased potential in repair of hydraulic structures. Consequently, research has been initiated to develop a procedure for underwater installation of geomembrane repair systems. The objectives of this REMR research are to (a) develop concepts for geomembrane systems that can be installed underwater to minimize or eliminate water intrusion through cracked or deteriorated concrete and defective joints, and (b) demonstrate the constructibility of selected concepts on dams and intake towers.

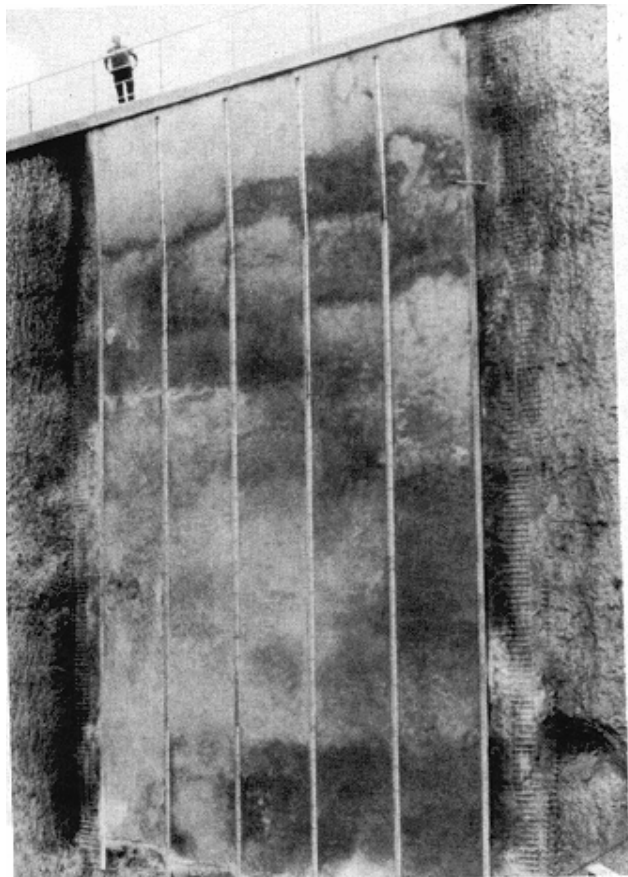


Figure 8-61. Steel profiles embedded in shotcrete, Cignana Dam



Figure 8-62. Installation of geocomposite, Pracana Dam

8-8. RCC Applications

The primary applications of roller-compacted concrete (RCC) within the Corps of Engineers have been in new construction of dams and pavement. Meanwhile, RCC has been so successful for repair of non-Corps dams that the number of dam repair projects now exceeds the number of new RCC dams. The primary advantages of RCC are low cost (25 to 50 percent less than conventionally placed concrete) and rapid construction. Guidance on the use of RCC in civil works structures is given in EM 1110-2-2006.

RCC has been used to strengthen and improve the stability of existing dams, to repair damaged overflow structures, to protect embankment dams during overtopping, and to raise the crest on existing dams. Generally, there are three basic types of dam repairs with RCC: (a) an RCC buttress is placed against the downstream face of an existing dam to improve stability; (b) RCC is used to armor earth and rockfill dams to increase spillway capacity and provide erosion protection should the dam be

overtopped; and (c) a combination of the first two, a buttress section and overtopping protection. In addition, RCC has been used to replace the floor in a navigation lock chamber, to help prevent erosion downstream of a floodway sill, and to construct emergency spillways. Selected applications of RCC in repair of a variety of structures are summarized in the following case histories.

a. Ocoee No. 2 Dam. This 9.1-m (30-ft)-high, rock-filled timber crib dam was completed in 1913, near Benton, TN. A riprap berm was placed on the downstream side of the dam in 1980 to improve the stability of the dam. However, flash floods during reconstruction resulted in four washouts of the rock and forced the agency to adopt another method for rehabilitation. The new solution was the first application of RCC to provide increased erosion resistance during overtopping (Hansen 1989). Approximately 3,440 cu m (4,500 cu yd) of RCC was placed in stairs on the downstream face of the dam (Figure 8-63). The dam has been subjected to periodic overtopping and, where the RCC was well compacted, it remains undamaged by water flow and weathering (Figure 8-64).

b. New Cumberland Lock. RCC was used to pave the floor of the lock chamber between the upstream emergency bulkhead gate sill and the upstream miter gate sill (Anderson 1984). A drop pipe was used to transfer the RCC from the top of the lockwall to the floor where it was moved by a front end loader, leveled with a dozer, and compacted with a vibratory roller (Figure 8-65). The RCC was placed in 0.3-m (1-ft) layers to a maximum thickness of 1.5 m (5 ft). The nominal maximum size aggregate (NMSA) was 25.4 mm (1 in.), and the cement

content of the mixture was 208 kg per cu m (350 lb per cu yd). A total of 1,645 cu m (2,152 cu yd) of RCC was placed at a cost of \$95 per cu m (\$73 per cu yd).

c. Chena Project. RCC was used by the U.S. Army Engineer District, Alaska, in lieu of riprap for erosion protection downstream of a floodway sill near Fairbanks. RCC was chosen, even though it was slightly more expensive than riprap, because riprap was in short supply in the area and RCC costs much less than conventionally placed concrete in Alaska (Anderson 1984). The RCC was placed in a section 1.5 m (5 ft) thick, about 12.2 m (40 ft) wide, and 610 m (2,000 ft) long. A total of 12,776 cu m (16,700 cu yd) was placed in 14 days. The cost of the RCC, including cement, was \$88 per cu m (\$67 per cu yd) compared to \$220 per cu m (\$168 per cu yd) for conventionally placed concrete on the same project.

d. Kerrville Ponding Dam. The dam consists of a compacted clay embankment 6.4 m (21 ft) high and 182 m (598 ft) long with a 203-mm (8-in.)-thick reinforced concrete cap. The dam, located on the Guadalupe River at Kerrville, TX, was completed in 1980. A service spillway to handle normal flows was created by lowering a 60-m (198-ft) section of the dam 0.3 m (1 ft). The entire dam was designed to be overtopped during flood condition; however, the dam sustained some damage when overtopped in 1981 and 1982. The worst overtopping, 3 m (10 ft) on New Year's Eve, 1984, damaged both the embankment and concrete cap on the downstream slope. The majority of the damage was to the service spillway where about 40 percent of the concrete facing was lost along with a significant portion of the embankment.

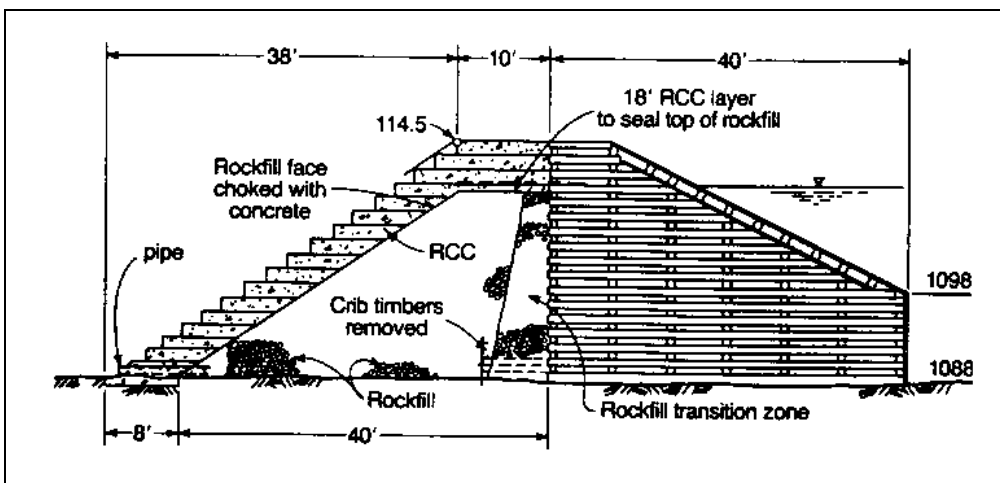


Figure 8-63. Section through modified dam, Ocoee Dam No. 2 (Hansen 1989)

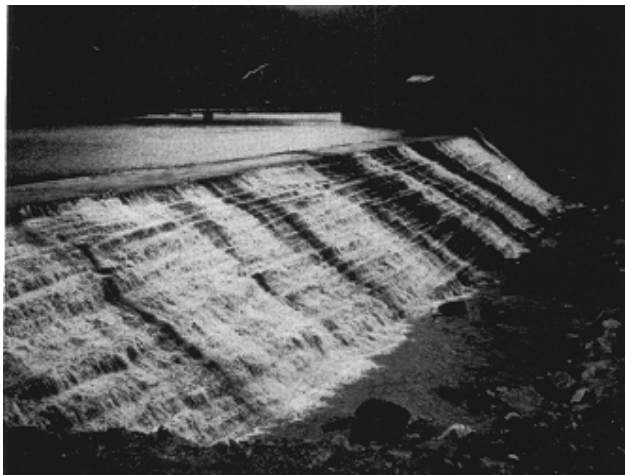


Figure 8-64. Condition of RCC following periodic overtopping, Ocoee No.2 Dam

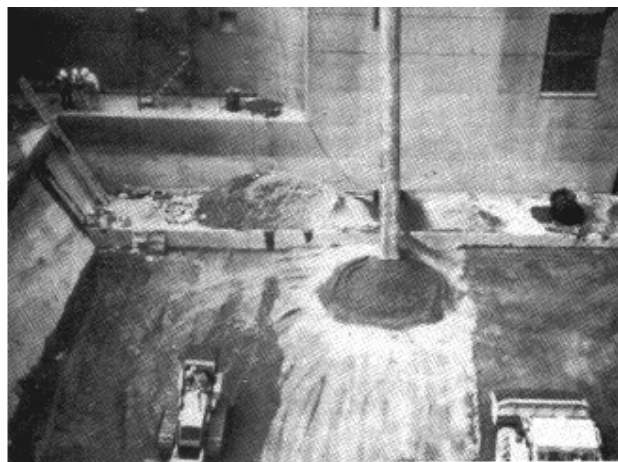


Figure 8-65. RCC placement, New Cumberland Lock (Anderson 1984)

(1) After several repair alternatives were evaluated, the fastest and most practical solution was to construct an RCC section immediately downstream of the dam (Figure 8-66). The downstream portion of the embankment was removed; the undamaged upstream portion acted as a cofferdam during construction of the RCC section. An 89-mm (3.5-in.) maximum size pit-run aggregate with 10 percent cement by dry weight was used at the base of the dam and in the last five 0.3-m (1-ft)-thick lifts. Cement content was reduced to 5 percent for the middle of the gravity section. The average compressive strength of the richer RCC mixture was 14.5 MPa (2,100 psi) at 28 days. Placement of the 17,140 cu m (22,420 cu yd) of RCC began in late June 1985 and was completed in about 3 months.

(2) About 1 month after completion of RCC placement, a 50-year flood overtopped the structure by 4.4 m (14.4 ft). Water flowed over the entire dam for about 5 days with a maximum flow of 3,540 cu m per sec (125,000 cu ft per sec). An inspection following the flood revealed only minor erosion of uncompacted material at the surface of the weir. Less than 2 years later, a 100-year flood overtopped the structure by 4.9 m (16.2 ft) with a maximum flow of 4,590 cu m per sec (162,000 cu ft per sec). Once again, the RCC performance was outstanding with only minor surface spalling.

e. Boney Falls Dam. The Boney Falls hydroelectric project is located on the Escanaba River about 40 km (25 mi) upstream of Lake Michigan. The dam, which was built during 1920-1921, consists of a 91-m (300-ft)-long right-embankment dam, a nonoverflow gravity dam, a three-unit powerhouse, a gated spillway with six tainter gates, a 61-m (200-ft)-long ungated spillway, and a 1,707-m (5,600-ft)-long left-embankment dam. Normal operating head on the power plant is approximately 15 m (50 ft).

(1) In 1986, it was determined that the original spillways were not adequate to pass the Probable Maximum Flood (PMF) under present-day dam safety criteria. After studying several alternatives, it was concluded that RCC overtopping protection for 305 m (1,000 ft) of the left embankment would be the least costly plan for increasing spillway capacity (Marold 1992). This plan was eventually modified so that an RCC gravity section would be placed immediately behind an existing concrete core wall in the embankment. This modification reduced the length of the RCC spillway section to 152 m (500 ft). The shorter length required the crest of the RCC gravity section to be lower to pass the same flow as the longer paved section. Therefore, the crest of the new RCC gravity section was set at 0.3 m (1 ft) below normal pool and a 1.2-m (4-ft)-thick fuse plug of erodible earthfill was placed on top of the RCC (Figure 8-67).

(2) The 3,710 cu m (4,850 cu yd) of RCC used in the spillway section was placed in 8 days at a cost of \$76/cu m (\$58/cu yd). When placement was completed, full-depth cores were taken from the spillway section for testing. Core recovery was 100 percent with 97-percent solid concrete and excellent bond between lifts. The average compressive strength of field-cast cylinders was just over 27 MPa (4,000 psi) at 28 days (PCA 1990).

f. Camp Dyer Diversion Dam. The dam, which was completed in 1926, is located on the Auga Fria River

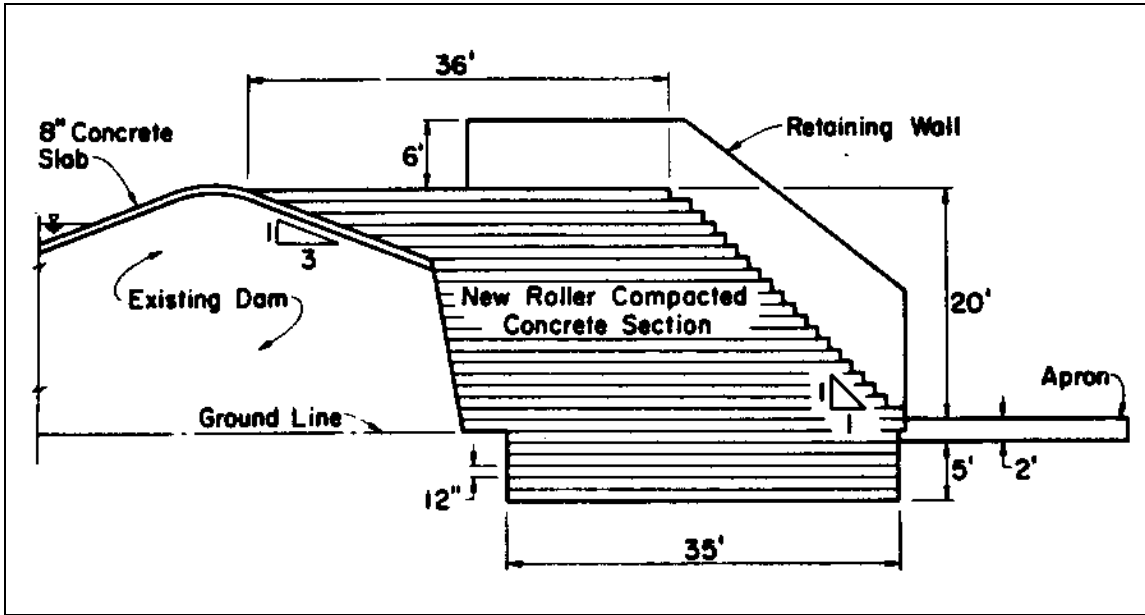


Figure 8-66. RCC gravity section, Kerrville Ponding Dam (Hansen and France 1986)

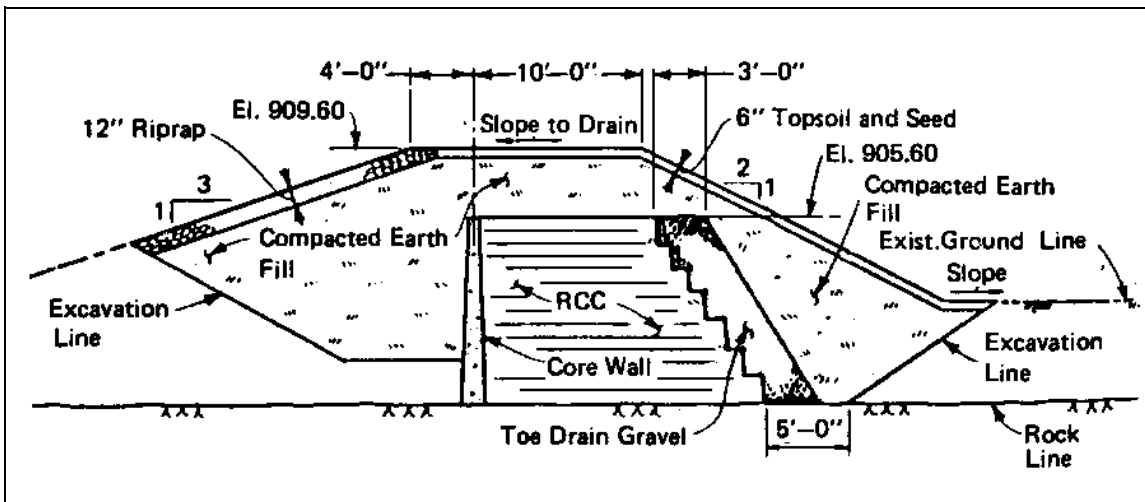


Figure 8-67. RCC spillway section, Boney Falls Dam (Marold 1992)

about 56 km (35 mi) northwest of Phoenix, AZ, and about 1.6 km (1 mi) downstream from Waddell Dam. The original masonry and concrete gravity structure had a crest length of 187 m (613 ft) and a maximum structural height of 23 m (75 ft). A smaller concrete gravity dike west of the dam had an 80-m (263-ft) crest length and a 7.6-m (25 ft) maximum structural height. Construction of the New Waddell Dam midway between the Waddell and the Camp Dyer Diversion Dams significantly reduced the storage capacity of the lower lake. Consequently, the height of the Camp Dyer Diversion Dam was raised

1.2 m (3.9 ft) to maintain the lake's original storage capacity, and the dam was modified to meet current criteria for static and dynamic stability of concrete gravity dams (Hepler 1992).

(1) To increase the dead load and the sliding resistance of the structure, an RCC buttress was designed for the downstream face of the existing dam. RCC was selected over conventional concrete because of its relative economy and ease of construction. In the original design, the buttress had a nominal width of 4.6 m (15 ft) and a

downstream slope of 8H:1V. However, in the final design, the width was increased to 6.1 m (20 ft) to accommodate two lanes of construction traffic on the RCC lifts. The RCC buttresses were capped with a conventional reinforced-concrete apron and ogee overflow crest (Figure 8-68).

(2) Approximately 11,800 cu m (15,400 cu yd) of RCC was used to construct the dam and dike buttresses. Total cost of the project, including the RCC buttresses and associated work, was about \$3 million.

g. *Gibraltar Dam.* The dam is located on the Santa Ynez River north of Santa Barbara, CA. The original dam was completed in 1922 and raised in 1948 to provide storage for the city's municipal water supply. It is a constant radius, concrete arch dam with a maximum height of 59 m (195 ft) and a crest length of 183 m (600 ft). The thickness of the arch varies from 2.1 m

(7 ft) at the crest to approximately 20 m (65 ft) at the base.

(1) The results of a 1983 safety evaluation indicated that the dam did not meet seismic safety standards. Therefore, an RCC buttress was constructed against the downstream face of the dam to alter the dynamic response characteristics of the dam and thus reduce the stresses induced during an earthquake. The addition of the gravity buttress effectively converted the existing arch into a curved gravity dam as shown in Figure 8-69 (Wong et al. 1992).

(2) The RCC method of strengthening was selected after three alternate methods were evaluated in terms of design, environment, construction, and cost. The other methods considered were placing a blanket of reinforced concrete or shotcrete over both faces of the dam, and constructing a rockfill buttress against the downstream

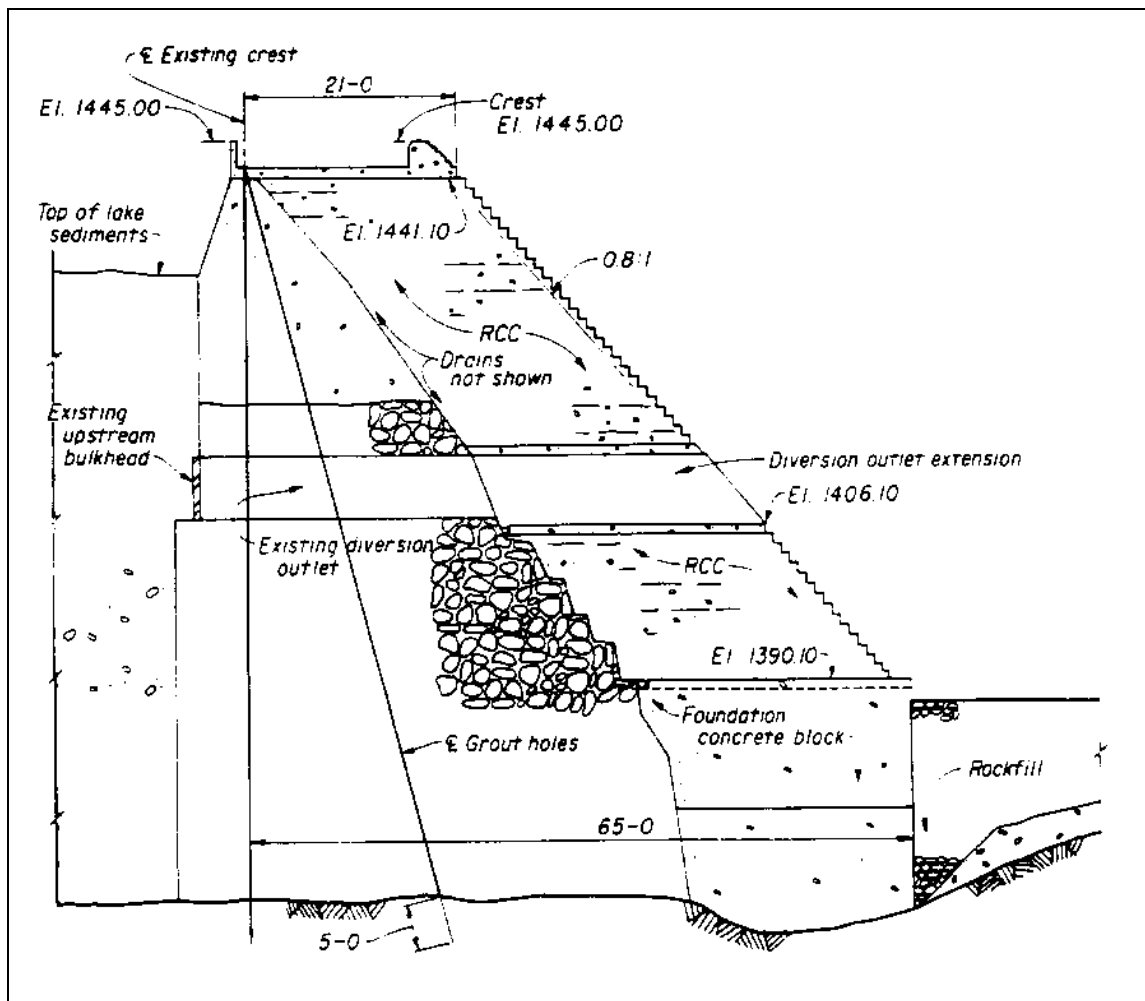


Figure 8-68. Cross section of modifications for Camp Dyer Diversion Dam (Hepler 1992)

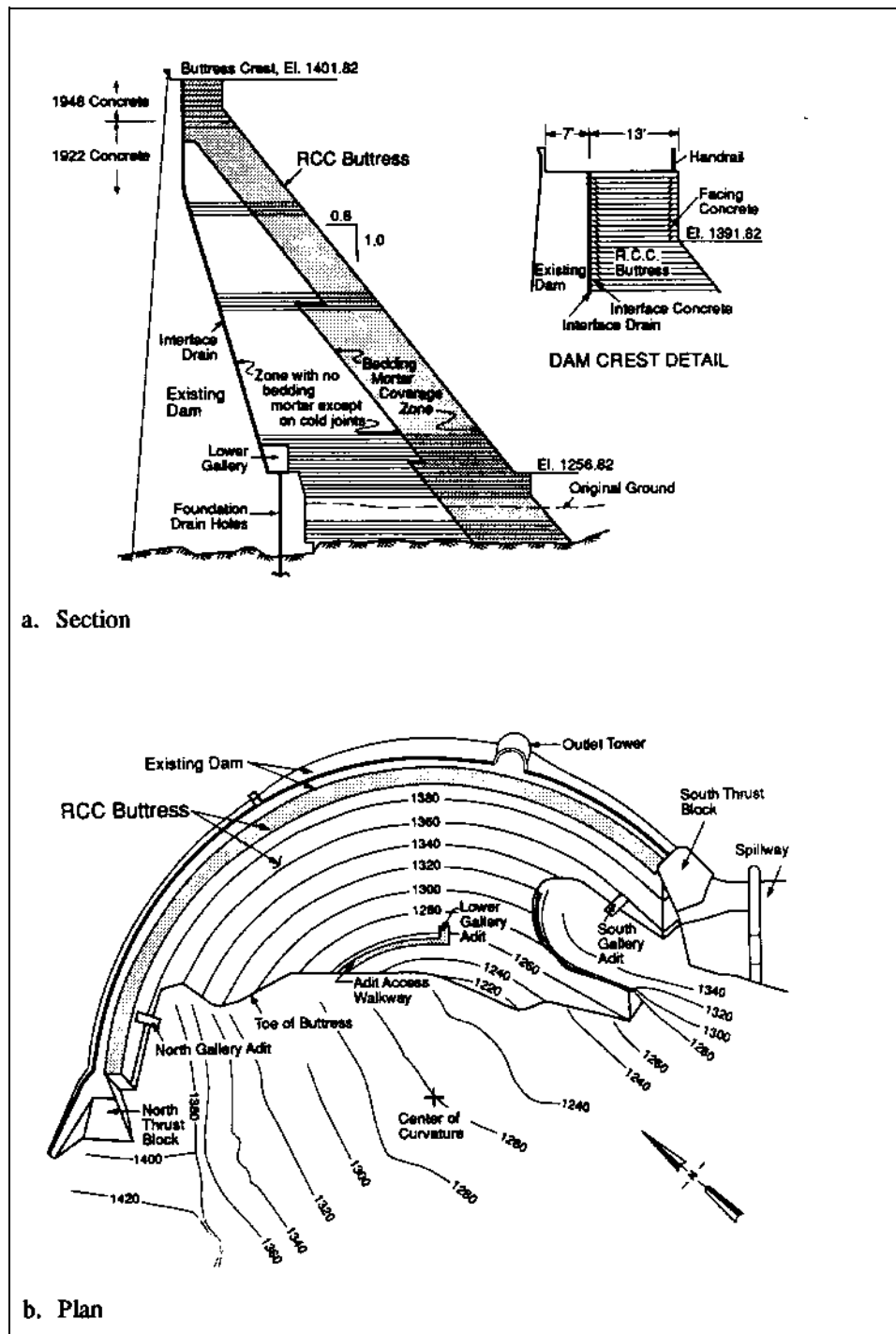


Figure 8-69. Section and plan of modified Gibraltar Dam (Wong et al 1992)

face. RCC was chosen because of the speed of placement that would translate to lower construction costs. The constricted wilderness site also made swift placement an advantage, and potential for environmental impact by other methods was a third factor in the choice.

(3) All onsite construction, including placement of 71,870 cu m (94,000 cu yd) of RCC, was completed in 1 year at a total cost of \$8.18 million.

h. Littlerock Dam. This multiple-arch, reinforced-concrete dam is located near Palmdale, CA, about 2.4 km (1.5 mi) south of the San Andreas fault. The historically significant dam, completed in 1924, has a maximum height of 53 m (175 ft) and a crest length of 219 m (720 ft). The mutipile-arch structure that comprises the main section of the dam consists of 24 arch bays with buttresses at 7.3-m (24-ft) centers. Results of stability and stress analyses showed that the dam did not meet required seismic safety criteria, principally because of a lack of lateral stability, a deficiency inherent in multiple-arch dams (Wong, Forrest, and Lo 1993).

(1) To satisfy preservation requirements and to provide adequate seismic stability, engineers designed a

rehabilitation program that included construction of an RCC gravity section between and around the downstream portions of the existing buttresses (Figure 8-70). Also, steel fiber-reinforced shotcrete with silica fume was used to resurface and stiffen the arches of the existing dam. RCC placement began in November 1993, and approximately 70,500 cu m (93,000 cu yd) was placed in just over 2 months. The RCC essentially converted the multiple-arch dam into a gravity structure. In addition, the crest of the dam was raised 3.7 m (12 ft) with conventional concrete to increase water storage capacity. The cost of the RCC buttress repair was \$12.8 million compared to an estimated cost of \$22.5 million for filling the arch bays with mass concrete.

(2) When the Northridge earthquake shook southern California in January 1994, the RCC placement was within 5 m (16 ft) of the crest of the existing dam. Although there was widespread damage to highway bridges within 56 km (35 mi) of the project, the quake had no noticeable impact on the dam.

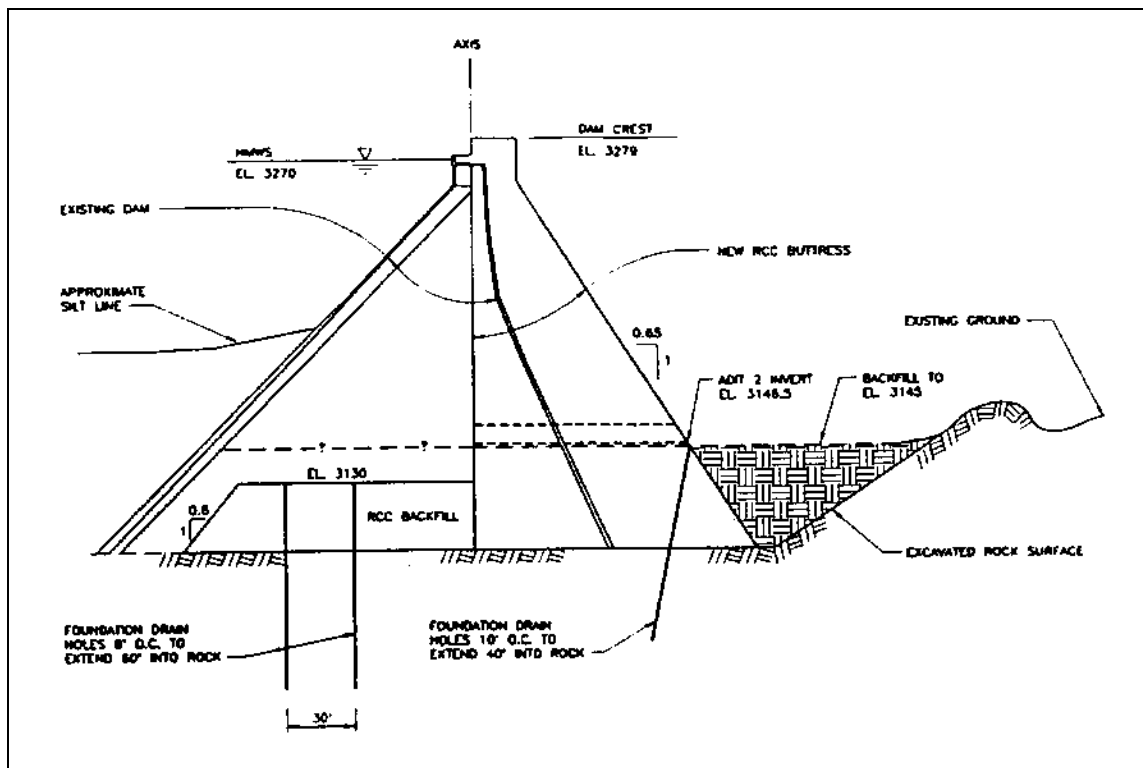


Figure 8-70. Littlerock Dam modification (Bischoff and Obermeyer 1993)